Characterisation and use of stabilised basecourse materials in transportation projects in New Zealand

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Abbreviations and acronyms

AWPT	area-wide pavement treatment
CBR	Californian bearing ratio test
CIRCLY	pavement analysis software (MINCAD systems)
CTB	cement-treated basecourse
ESA	equivalent standard axle
FBS	foamed bitumen stabilisation
FWD	Falling Weight Deflectometer
FWP	Forward Work Programme
ITS	indirect tensile test
MESA	million ESA
NZTA	New Zealand Transport Agency
PF	the power law factor
RF	the project reliability factor
RP	highway route position
SF	shift factor
SH	state highway
SME	subgrade modulus exponent
UCS	unconfined compression test
WMAPT	weighted mean annual pavement temperature

Contents

Executive summary

The objective of this research project was to provide guidance to the wider transport industry on the characterisation and effective use of stabilised granular materials in transportation projects, based on actual performance data from stabilised pavement solutions in New Zealand. The research was carried out between December 2009 and August 2011. The research team assembled into a comprehensive data inventory the performance data from New Zealand road pavements that utilised a range of stabilising agents and material types.

They then used the data in the inventory to investigate the characterisation of the stabilised materials, based on material strength data – notably unconfined compression strength (UCS) test data, and to a lesser extent, indirect tensile test (ITS) test data. The spread of material strength data, particularly the UCS data, was reasonably large, and based on varying sample compaction levels. The reported binder contents were low (generally <3% by dry mass of aggregate material). In the majority of cases the reported UCS results exceeded the expected range of UCS for modified materials (of between 0.7 and 1.5MPa).

We concluded that in the vast majority of cases, the stabilised granular materials in the project sites were behaving as bound materials (albeit lightly bound) rather than modified (unbound) materials. Our field studies included the collection and retrieval of intact cores of the stabilised granular materials from 17 project sites. Subsequent laboratory testing of these cores (UCS and ITS) verified lightly bound behaviour.

From the extensive data inventory for cement-stabilised sites, we were able to investigate the ongoing performance of a number of project sites. In some cases we could confirm the timing of 'premature failures including cracking and rutting' in the stabilised granular layer, and compare the reported outcomes with existing theory. We already suspected that the Austroads fatigue criterion for cement-bound materials with layer moduli <10,000MPa was conservative with respect to design expectations using cement-stabilised pavement layers with low cement contents (<3% cement by dry mass).

Our analysis of the inventory data, including pavement sites where the ongoing performance had either exceeded that predicted by Austroads or in contrast, had exhibited premature failure in the stabilised granular layer (e.g. cracking or rutting, or both) enabled us to prepare a *conceptual pavement performance model for cement-stabilised pavements*. We then used this model to predict the observed performance of pavements using stabilised granular materials that behave as lightly bound layers.

Our conceptual pavement performance model for cement-stabilised materials has more coefficients than the current Austroads equation, but is no more time-consuming to invoke in practice, and allows greater flexibility in matching observed data. The inverse equation needs to be calculated iteratively at present, and utilises a cured layer modulus E_C. We believe this model could be enhanced in future as more pavement performance data becomes available.

At the time of writing, we concluded that in New Zealand there was only a limited data pool for foamed bitumen/cement-stabilised (FBS) project sites, a maximum of five years' service life, and a lack of identified failure modes, which all prevented us from confirming a conceptual performance model for FBS.

The design of pavements incorporating cement- and FBS-stabilised basecourse layers should continue to use the guidelines provided by Transit NZ, and also include a check to ensure that:

- for cement-stabilised pavements the layer stress levels are < 50% of the tensile strength of the material, and layer strains are also managed in accordance with observed behaviour
- for FBS-stabilised pavements the modular ratio of the FBS layer to the underlying sub-base layer modulus is not more than 5.

Abstract

The stabilisation of near-surface granular pavement materials is accepted practice in transportation maintenance and capital development projects in Australasia. Stabilisation in this context involves the mechanical introduction of reactive agents, including cement and foamed bitumen, into existing or manufactured granular materials, with or without existing seal inclusion.

Present-day design guides characterise stabilised granular materials as either modified or bound, depending primarily on the amount and type of reactive agent used in the stabilising process. Modified materials are modelled as unbound granular materials in a pavement.

Bound (cemented) materials are modelled as layers with tensile load-carrying capacity within the pavement. Cracking in the bound pavement layer is governed by 'fatigue relationships'.

This research has shown that stabilisation with smaller reactive agent contents (<3% by dry mass) can deliver materials that should be modelled as lightly bound, delivering cost-effective pavement solutions.

This research report describes the collection and interrogation of performance data from New Zealand road pavements that utilised stabilised granular materials. The research, carried out from December 2009 to August 2011, compared actual stabilised pavement performance with the expectations in published design guidelines. A conceptual pavement performance model for near-surface 'lightly bound' stabilised granular pavement layers that better matches observed pavement behaviour is proposed.

1 Introduction

1.1 Background

The stabilisation of near-surface granular pavement materials is a widely used practice in transportation maintenance and capital development projects in New Zealand. *Stabilisation* in this context involves the introduction of reactive agents, such as cement, lime and foamed bitumen, by mechanical means into existing or manufactured granular pavement materials, with or without seal inclusion (see figure 1.1).

Figure 1.1 Stabilisation of near-surface granular layers

Pavement designers, contractors and asset managers use stabilisation methods to improve the long-term performance and load-carrying capacity of pavement materials and structures. The process of stabilisation enables the following changes within granular materials:

- reduces in-situ water content and therefore susceptibility to strength reduction due to wetting in sensitive materials
- reduces the plasticity in cohesive materials
- binds together the particles in the granular material, producing a 'stronger', more resilient near-surface pavement layer.

1.2 Research objective

The objective of this research project was to provide guidance to the wider transport industry on the characterisation and effective use of stabilised granular materials in pavement projects, based on actual performance data from stabilised pavement solutions in New Zealand. The project included specific field research into pavement sites around New Zealand where performance data was available. By collecting and interrogating performance data from road pavements that utilised stabilised granular materials (including a range of stabilising agents and material types) in various locations around the country, the research team studied how actual stabilised pavement outcomes compared with the expectations based on current recommended design and construction practices. The research team sought to answer the question 'Have the design expectation(s) at these sites translated into observed field performance?'

Stabilisation can be used to improve the performance of various materials in a pavement structure, such as the foundation or subgrade. This research, however, is focused on the *stabilisation of the granular layers at or near the pavement surface within a pavement, with or without seal inclusion*.

1.3 Research context

Austroads (2006) categorises 'stabilised granular materials' as granular, modified or bound, as shown in table 1.1 (which is reproduced from table 3.1 in Austroads 2006).

These stabilisation categories, which have been routinely based on established laboratory tests such as the unconfined compression test (UCS), indirect tensile strength test (ITS) and the Californian bearing ratio test (CBR), have significant implications in terms of:

- design
- construction and failure mode expectations
- the long-term behaviour of pavements and load-bearing layers that incorporate stabilised granular materials.

Table 1.1 Types of stabilisation as defined by Austroads (2006)

Category of stabilisation	Indicative laboratory strength after stabilisation	Common binders adopted	Anticipated performance attributes
Subgrade	CBR[1] > 5% (subgrades and formations)	▪ Addition of lime ▪ Addition of chemical binder	▪ Improved subgrade stiffness ▪ Improved shear strength ▪ Reduced heave and shrinkage
Granular	40% < CBR[1] < +100% (subbase and basecourse)	▪ Blending other granular materials which are classified as binders in the context of this Guide	▪ Improved pavement stiffness ▪ Improved shear strength ▪ Improved resistance to aggregate breakdown
Modified	0.7 MPa < UCS[2] < 1.5 MPa (basecourse)	▪ Addition of small quantities of cementitious binder ▪ Addition of lime ▪ Addition of chemical binder	▪ Improved pavement stiffness ▪ Improved shear strength ▪ Reduced moisture sensitivity, i.e. loss of strength due to increasing moisture content ▪ At low binder contents can be subject to erosion where cracking is present
Bound	UCS[2] > 1.5 MPa (basecourse)	▪ Addition of greater quantities of cementitious binder ▪ Addition of a combination of cementitious and bituminous binders	▪ Increased pavement stiffness to provide tensile resistance ▪ Some binders introduce transverse shrinkage cracking ▪ At low binder contents can be subject to erosion where cracking is present

Notes:

1. Four day soaked CBR.
2. Values determined from test specimens stabilised with GP cement and prepared using Standard compactive effort, normal curing for a minimum 28 days and 4 hour soak conditioning.

In mechanistic pavement design, granular materials are modelled as unbound layers. They are considered to be cross-anisotropic, have a Poisson's Ratio of 0.35, and need to be sub-layered when modelled mechanistically using software such as CIRCLY5.[1]

Modified materials are considered to behave as unbound granular materials in a pavement, and are characterised and modelled in the same manner. Modified materials incorporating reasonably well-graded granular materials and reactive agents such as cement can be further characterised as having ITS strengths of equal to or less than 80kPa, UCS (seven days' moist-curing at standard compaction) of equal to or less than 0.8MPa, and/or resilient modulus ranging from 700MPa up to 1500MPa – the latter contrasts with the range of resilient modulus for high-standard unbound crushed rock of 300–700MPa.

Typically, in New Zealand a premium M/4 basecourse (well-graded crushed granular material with a maximum stone size of 40mm and used in the near-surface layers of an unbound pavement) would be modelled using a sub-layered modulus of up to 450–500MPa.

A *premium* modified material can be expected to provide enhanced performance and will usually be modelled as a premium M/4 basecourse. The behaviour of the basecourse layer can be affected by rutting and shallow shear at or near the surface.

In contrast, bound (cemented) materials behave as 'beams', or layers, with tensile load-carrying capacity within the pavement. They are considered to be isotropic, have a typical Poisson's Ratio of 0.2, and for modelling purposes, have a resilient modulus that depends primarily on the amount and type of reactive agent used in the stabilising process.

The performance of a bound material is governed by a 'fatigue relationship' that controls cracking within the bound layer. Typically, the fatigue relationship takes the form shown in the equation below:

$$N = RF\left[\frac{a}{\mu\varepsilon}\right]^{b}$$

(Equation 1.1)

Constants *a* and *b* above are dependent on the material, *N* is the number of passes of the design traffic, and $\mu\varepsilon$ is the critical tensile strain in the pavement layer in question. RF is the reliability factor.

Austroads (2008) describes the fatigue criteria for cemented materials with layer moduli between 2000MPa and 10000MPa in figure 6.5. The reliability factor RF is described in table 6.8 of the same reference.

When utilising stabilised granular materials in a new pavement or pavement maintenance project, the current recommended practice for a pavement designer is to assume that the stabilised layer will behave as either modified or bound. The performance of the pavement will then depend on whether the design assumptions translate into field performance. The consequences of 'getting it wrong' can be significant.

In a new pavement project, if a 'bound' stabilised material does not achieve the required strength and durability, it is likely that the overall pavement will not have sufficient load-carrying capacity (e.g. depth to foundation or subgrade) and therefore will crack prematurely and need more maintenance intervention (and strengthening) earlier in the pavement life cycle. At another extreme, if a stabilised material is assumed to only be modified, but is in fact bound, then premature fatigue cracking can occur, particularly in thin pavement layers. This will again require early unplanned maintenance intervention. It will also

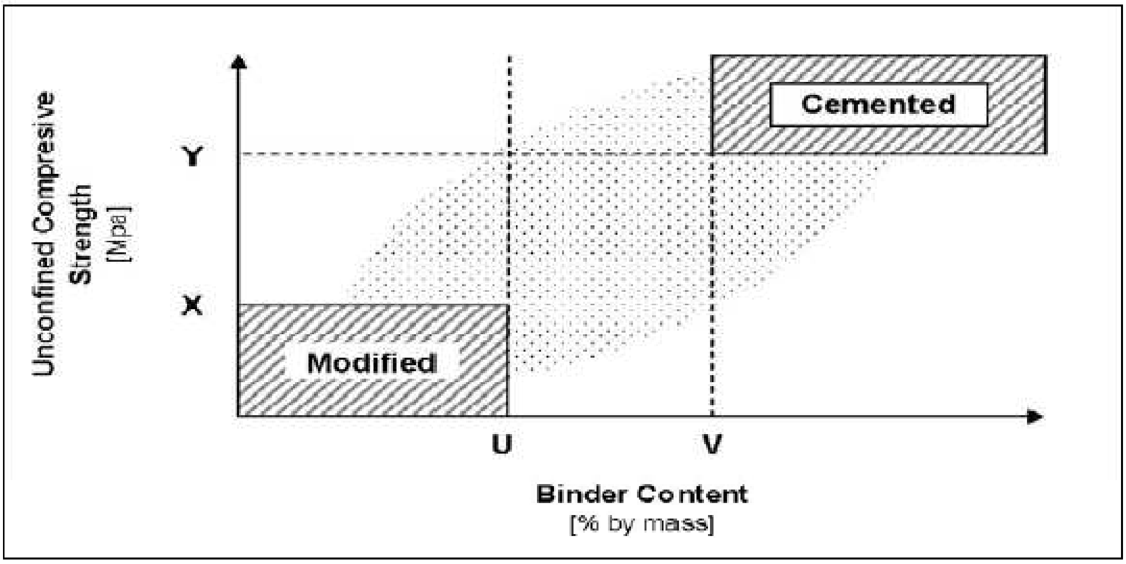
1 CIRCLY5, MINCAD Systems

probably change the overall behaviour of the pavement, and may require more expensive maintenance intervention, including crack sealing, or in the worst case, complete removal and replacement of the recently stabilised material.

Near-surface granular material stabilisation of existing pavement materials is a widely used maintenance practice throughout New Zealand. The use of small portable stabilisers has enabled contractors to use this method of repair on an ever-expanding list of projects and project locations. The consequence of 'getting it wrong' in one smaller, isolated pavement repair may not appear significant on its own. But the consequences of an increasing number of stabilised pavement repairs, or individual larger repairs, failing prematurely would be expensive rework and wasted resources.

There is uncertainty about how to adequately describe the theoretical boundaries between modified and bound materials in a manner that can be used effectively in practice by pavement designers and contractors. In figure 1.2 below we show pictorially how this 'uncertainty' can be described when considering two commonly used parameters to characterise stabilised materials: binder content (the amount of reactive agent added to the stabilised material) and UCS.

Figure 1.2 Characterisation of modified (unbound) and cemented (bound) materials



In figure 1.2, the lower hatched area outlined by X and U (lower binder content and UCS) describes material properties currently recognised by Austroads and others as unbound materials. These materials are assumed to behave as unbound materials, and the performance of pavements incorporating such modified materials would be associated with predicted rutting or consolidation, and/or shallow shear of the near surface under the load.

The upper hatched area outlined by Y and V (higher binder content and UCS) describes material properties recognised by Austroads and others as bound materials. Provided that fatigue cracking in a bound layer is controlled, these materials can deliver low-maintenance pavement solutions.

The lightly shaded area in figure 1.2 describes pictorially the current 'zone of uncertainty'. Here, the combination of material type (including aggregate mineralogy), binder content and layer depth has the potential to result in a pavement layer that could, at either extreme:

- initially behave as a bound layer, then suffer early fatigue cracking, premature failure, and the need for unplanned maintenance intervention or:

- behave as a modified layer only (not bound) that cannot carry the design loads in the manner expected, and requiring early intervention and probably strengthening.

The difficulty that remains for designers/contractors and asset managers is that the current methods recommended for characterising modified and/or bound materials during the design process are not confidently based on 'actual performance data' under New Zealand conditions. The benefits of building with stabilised materials can be lost if the designer cannot properly characterise and justify the behaviour of the pavement.

This research project provides wider field validation of stabilised material properties under New Zealand conditions. This will in turn enable practitioners to confidently use the most suitable stabilised material (modified or bound) for individual project solutions.

1.4 Research project methodology

This research project was completed in the following three steps:

1. Collecting, collating and reviewing pavement-condition and performance information from numerous project sites throughout New Zealand:
   The types, locations and conditions of these pavements varied significantly. All were stabilised using either cement or foamed bitumen/cement. Once the data had been collected, each pavement section was back-analysed using ELMOD[2] to provide information about representative stabilised layer modulus and tensile strain at the base of the stabilised layer. In some cases we were also able to relate the back-calculated modulus/strain information to actual 'failure' conditions on site, based on our observation of defects during the on-site testing (see step 2) and feedback from the road-controlling authority involved. The data was then collated in a pavement inventory that is being maintained by Tonkin & Taylor.
2. Obtaining performance data using the falling weight deflectometer test (FWD) and machine pavement-coring tests from pavement sites in the North Island:
   Subsequent laboratory-based ITS tests on the retrieved cores, and back-analysis of the site-specific FWD data, provided information about the stabilised layer modulus and tensile strain at the base of the stabilised layer at these sites.
3. Evaluating the data and analysis information we had gathered, investigating how this data could assist with future planning and design of granular stabilisation projects, and reporting our findings and conclusions in this report.

2 ELMOD Software with analysis completed by Tonkin & Taylor, Dunedin

2 Background: pavement area treatment using stabilisation in Gisborne and Hawke's Bay state highways

2.1 Introduction

Over the last 15 years, area-wide pavement treatment (recycling) projects in the New Zealand Transport Agency's (NZTA) regions 5 and 6 (Gisborne and Hawke's Bay) have utilised upper-pavement-layer stabilisation techniques. This work has been described as 'pavement recycling' (Opus International Consultants 1998)

In this context, recycling originated from the need to provide least whole-of-life-cost pavement maintenance options for pavements with unstable multiple-seal and near-surface pavement layers. The need for recycling was usually triggered by pavement maintenance performance data that identified declining seal life and increased surfacing maintenance costs in ageing pavements.

The earliest recycling works were completed in the 1995/1996 construction season. In these early days, the pavements being treated usually had binder-rich multiple seal layers (often with a combined layer depth of 50–75mm). The first trial recycling treatments involved cold pulverising and then relaying the existing asphalt-rich surfacing layers without any other additives. This usually proved unsuccessful because the pulverised bitumen did not mix evenly. Consequently the recycled layer did not hold together well. Once cement binder was added to the recycling process, the process proved to be very successful. This process spread to other networks around the country, notably Waikato, Bay of Plenty and Northland, with similar success.

2.2 Investigation and design approach

Recycling/stabilisation of the near-surface pavement layers was, in this context, intended to produce a new modified layer that had a design life of 10 years or more, and provided a sound, homogeneous layer to support future overlay (Gray and Hart 2003). The investigation and design approach, which has been followed consistently over 16 years, recognises the challenge of replicating, in small-scale laboratory or field tests, the outcomes of modifying existing pavement materials with cement using a powerful pulverising plant. It also recognises the need to ensure that the new modified pavement layer is being constructed as part of an otherwise-sound pavement that is not being adversely affected by deep-seated problems (Gray 2008).

In other words, the investigation and design approach that continue to be employed seeks to ensure that the problem or problems being addressed by the recycling process are indeed founded within the near-surface pavement seal and aggregate layers.

The flowchart in table 2.1 below describes the investigation and design steps that have been used to support the successful and ongoing recycling programme in the Hawke's Bay and Gisborne regions.

Table 2.1 Flowchart of investigation and design steps used to support recycling

	Description of investigation and design process	Key inputs
Step 1	• Select the walkover site in Forward Work Programme (FWP) • Plan FWD investigations. • Identify signs of pavement distress.	From records, identify the length and location of site, traffic data and likely pavement structure, to assist preliminary FWD analysis.
	⇓	
Step 2	• Undertake FWD survey of each traffic lane. • Complete preliminary elastic back-analysis using the assumed pavement structure.	Use recorded pavement deflections from FWD, known traffic data and assumed/known pavement structure.
	⇓	
Step 3	• Review recommendations for the site from the FWD analysis. • Programme testpit excavations at selected locations in low- and high-strength areas.	Look for indicators of low or variable layer moduli, high deflections (>1.5mm).
	⇓	
Step 4	• Complete testpit investigations and pavement material identification. • Sample for laboratory tests if required eg bitumen content, cement reactivity.	Identify seal and pavement depth and structure, material types, subgrade material and strength (Scala Penetrometer).
	⇓	
Step 5	• Re-run the elastic back-analysis of the pavement using the FWD deflection data and known pavement structure.	Use all existing investigation data.
	⇓	
Step 6	• Review the conclusions from the elastic back-analysis – is the site still suitable for recycling? • If it is, confirm what treatment is required. • If it is not, recommend another treatment.	Confirm if recycling/stabilisation will solve the observed pavement 'problems' and design solutions for each site.
	⇓	
Step 7	Construction and as-built monitoring: • During construction, review the as-built QA data to check that the as-built pavement achieves what was expected at design stage, and utilise the findings to review the design philosophy.	As-built monitoring to include UCS testing, condition rating and FWD survey and elastic back-analysis.

The investigation and design process at step 6 is a 'go or no-go' decision point. If at step 6 a treatment site does not appear to suit 'recycling', then the designer can recommend that the treatment should not proceed. Alternatively, the designer can consider how the site could utilise stabilisation construction options, but using another design outcome, including the following:

- For pavements that appear to be exhibiting distress caused by soft subgrade and/or inadequate pavement depth, the existing pavement can be overlaid with make-up metal prior to the top-surface stabilisation process, thus providing improved cover and reduced strains applied to the subgrade and lower pavement layers.

- A granular overlay prior to top-surface layer stabilisation can also be used to improve the granular make-up and properties of the stabilised layer, giving a robust substrate for sealing.
- If the pavement appears to be exhibiting distress caused by soft subgrade and inferior-quality lower-pavement materials, then the existing pavement can be stabilised, and then the stabilised layer overlaid with either unbound basecourse or modified basecourse products. This approach improves both the depth and consistency of the overall pavement, and improves the cover above the soft subgrade.
- In all cases, improvements to subsoil and surface drainage should occur if required. The subgrade modulus exponent (SME) output from the FWD back-analysis has been shown to be a useful tool to identify ‘at-risk’ subgrade moisture conditions. In this context, a low SME (<0) in the typical subgrade conditions found in these regions (softer sedimentary silt, or silty sand) was often found to be associated with higher in-situ moisture contents. Drainage improvements in such conditions were therefore considered to be affordable ‘pavement performance insurance’.

In summary, the underlying philosophy in this investigation and design approach was to investigate, at a project level, the likely causes for observed maintenance defects, and to determine whether the stabilisation of the top-surface layers (including seal with or without make-up metal) would address the probable causes for these defects. In section 5 of this report, we consider whether this approach could now be improved, based on the research findings described here.

3 Existing stabilised pavement review

3.1 Data collection and pavement back-analysis

The research team contacted interested parties and known associates throughout New Zealand to request information about pavements developed or rehabilitated using stabilised near-surface granular layers. The response was very encouraging. Design and pavement performance data was received from sites including:

- the North Island East Coast (Hawke's Bay and Gisborne)
- Northland and Auckland
- Waikato and Bay of Plenty
- Central Otago.

The data included:

- pre-construction investigation and design data, including test-pit and FWD data
- post-construction performance data (FWD) and condition data (roughness, cracking etc).

The names and locations of the project sites have, where requested, been kept anonymous to retain project confidentiality.

The project data was then assembled into a database, and back-analysis of the likely pavement conditions was undertaken by David Stevens and Graham Salt of Tonkin & Taylor, using the software programme ELMOD.

In our ongoing analysis, we separated the cement-stabilised sites from the foamed bitumen/cement-stabilised (FBS) sites (refer to section 5).

In figures 3.1 and 3.2 we present the back-calculated performance criteria (modified layer modulus and tensile strain at the bottom of the stabilised layer) from the cement-stabilised and FBS project sites reported to us in part 1 of this project.

The data presented below does not attempt to differentiate between sound or unsound pavement. In figure 3.1 we inserted on the plot the 'expected' crossover between cement-modified and cement-bound pavement behaviour (based on current thinking).

As expected, owing to increased stiffness the tensile strain at the bottom of the stabilised layer decreased as the stabilised layer modulus increased.

We were encouraged by the 'grouping' of the data. It appeared to us that if we were able to identify sites where we could confirm the time (and hence past traffic loading) to 'pavement failure' (eg cracking, rutting, shallow shear and shoving in the stabilised layer), then it would be possible to identify the modulus/strain conditions near or at failure.

Figure 3.1 Cement-stabilised project data

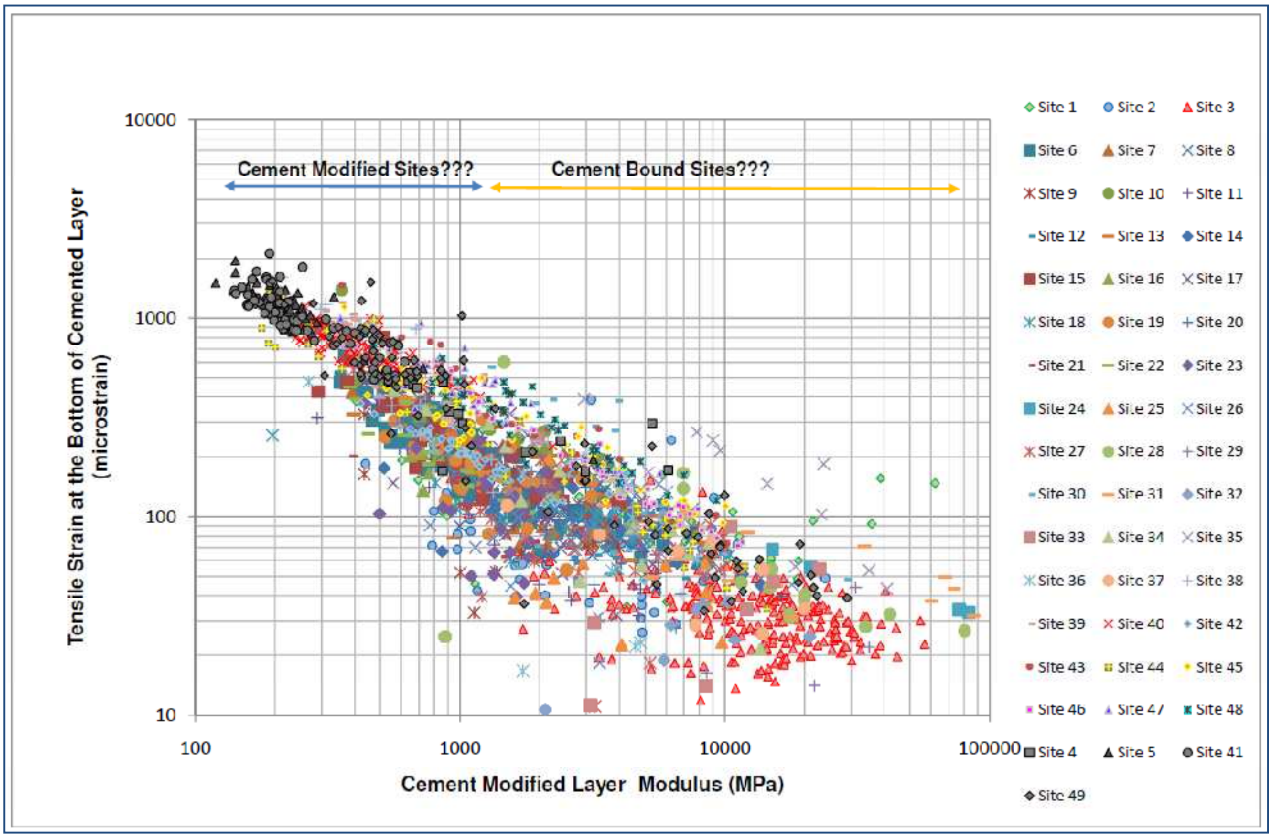
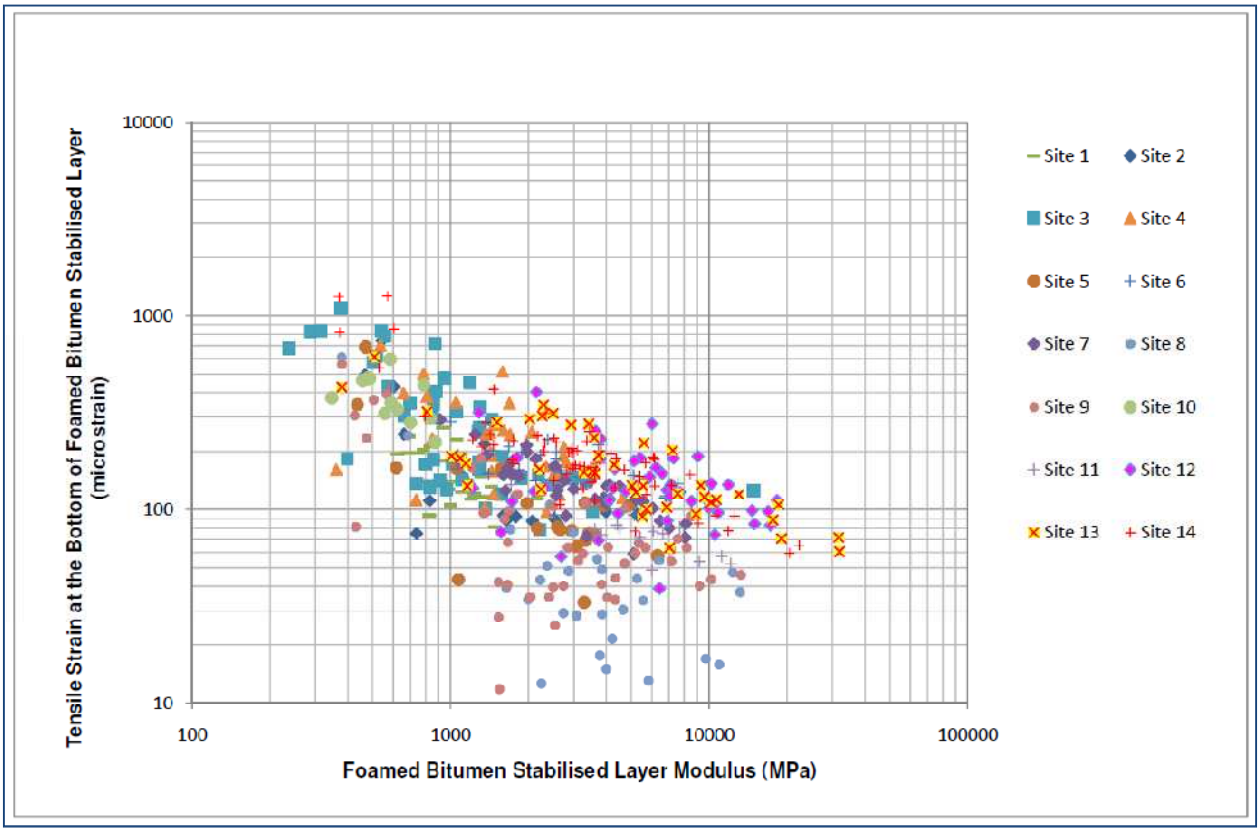


Figure 3.2 FBS project data



Only isotropic modulus values are presented, firstly because they are able to be generated very rapidly using the ELMOD software, and secondly because this enables presentation of all types of materials as a continuous function of modulus (ranging from those with so little binder that they are virtually unmodified, through modified and including up to heavily bound materials).

In practice, most sites include at least two of the categories in the aggregate stabilisation spectrum (shown earlier in table 1.1) and a number cover all three. Therefore until a better fundamental understanding of stabilised materials in New Zealand is obtained, the refinement of allocating a precise modulus value at which a material becomes bound (and hence should be modelled with an anisotropic modulus) has been left to a subsequent investigation.

3.2 As-built stabilised material strength data interrogation

We used the data to investigate how the design expectations at these sites had translated into actual field performance. In most of the sites, the stabilised granular pavement layers had been assumed by the designers to be ‘modified’. The following general observations were made:

- Granular pavement materials were stabilised using either cement or foamed bitumen/cement additives, with the great majority of these layers being the top-surface layer (immediately beneath the surfacing).
- Most of the pavements reported to us were performing as expected, requiring only routine maintenance and no major interventions (to date) to correct defects such as rutting or extensive cracking. Some sites had ‘failed’ and have been used as ‘markers’.

- The pavement ages varied – some cement-stabilised pavement sections were nearly 15 years old, while the FBS sites evaluated for this project were, at most, only 5 years old.

To recap – a modified stabilised material is expected to report a 28-day UCS between 0.7MPa and 1.5MPa. Our project data included reported UCS strength data from the investigation/design and/or construction activities. In the latter case, UCS samples were frequently taken from behind the hoe and compacted in moulds on site, using heavy compaction.[3] In some cases, UCS results for cement-modified materials were estimated from the Californian bearing ratio test data using the relationship UCS = CBR/100.

In figure 3.3 we report the UCS vs binder content comparison for the various data in the inventory. We have provided some guidance as to location of the sites in the legend. For the foamed bitumen/cement-modified sites, the binder content was taken as the combined sum of the foamed bitumen and cement – for example, in figure 3.3 the combination of 3.5% foamed bitumen plus 1% cement is represented by a total binder content of 4.5%. The data points labelled 'Auckland Lab Data from May 2011' were obtained using the core samples from this project's field investigation.

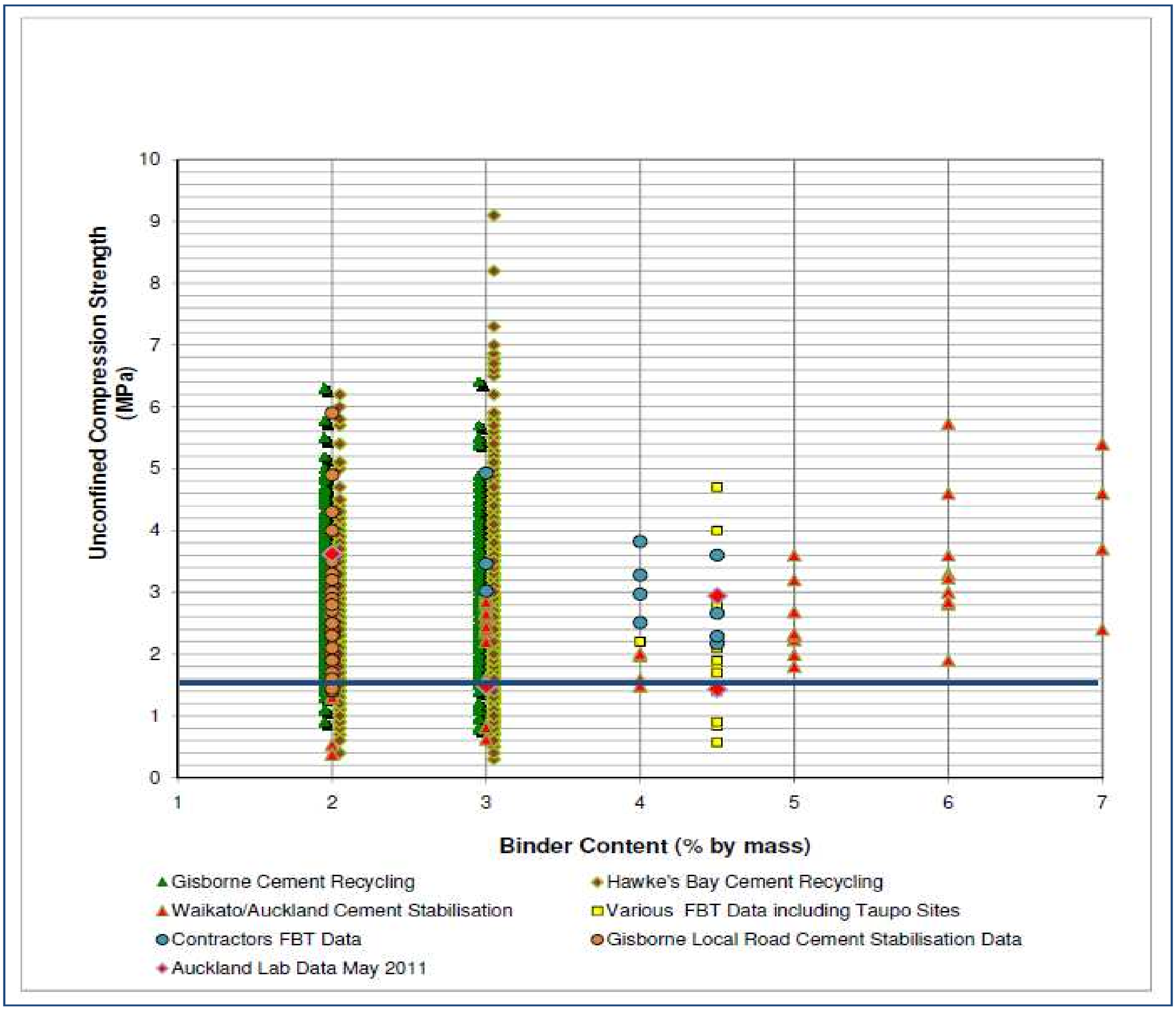
The horizontal blue line at a UCS of 1.5MPa in figure 3.3 is the currently accepted upper limit for modified materials. If we consider a UCS of greater than 3MPa as representing a true 'bound' layer, then the majority of the pavement inventory data falls into the 'bound' category.

Further notes about figure 3.3 are as follows:

- The data from the Gisborne and Hawke's Bay recycling projects reports cement stabilisation of the top pavement layers to depths of 200–250mm. Most of the UCS data shown is construction data derived from test samples compacted on site into CBR moulds using the heavy compaction hammer. The materials stabilised were mostly sedimentary greywacke or siltstone in origin, and included bitumen-bound pavement materials.
- The data reported for Waikato/Auckland cement stabilisation is mostly from pre-construction testing using a variety of aggregates of sedimentary and volcanic origin.
- The sites described as FBS are pavements stabilised with foamed bitumen/cement (usually 3–3.5% foamed bitumen and 1–1.5% cement). The stabilised materials include volcanic dacite, metamorphic schist (from the Crown Range) and sedimentary greywacke, based on both pre- and post-construction testing.

3 NZS 4402 1986: Test 4.1.2

Figure 3.3 Unconfined compressive strength and binder content comparisons for stabilised project sites



The UCS data presented in the above figure was prepared in a variety of ways (usually either heavy compaction or vibrating hammer compaction levels). According to Austroads (2006), the UCS limits for modified materials are based on a standard compaction effort. The difference in compaction effort will have contributed in part to the higher reported UCS outcomes in New Zealand.

4 Field study

4.1 Introduction

The project data we received from colleagues around New Zealand represented stabilised granular pavement sites covering a wide range of geological, geographic and pavement material conditions. Our planning for the field study involved identifying, from all the data received, project-specific information relating to a range of sites that we could visit in the field.

Our field study then sought to obtain the following information from each site:

- visual-inspection-based pavement condition data
- existing pavement structural data using the FWD
- core extraction using compressed air/water drill-coring techniques
- laboratory-strength testing of intact cores.

The cores obtained from each site were then returned to the Napier and Auckland Opus Engineering Laboratories for material characterisation and destructive testing (UCS and/or ITS testing).

The field testing (FWD and pavement coring) was carried out concurrently on each site. This enabled the FWD operator and our field staff to locate and test areas of pavement in variable condition (where possible, both sound and unsound conditions), rather than only test at prescribed chainages along the road. The field study work was completed in July/August of 2010.

4.2 Field testing

The field testing was undertaken by staff from the Opus Engineering Laboratory in Napier, with specialist support from Tonkin & Taylor (FWD) and Downer Engineering (pavement coring and traffic control). The team started from the southernmost location (site 15 in table 4.1 overleaf) and worked north, finishing at site 1.

The photographs in figure 4.1 show how the field core sampling was typically undertaken.

Figure 4.1 Photographs showing pavement core recovery technique

The FWD test rig is shown in figure 4.2. The testing was completed in or very near the outer wheeltrack, in alternating directions.

Figure 4.2 FWD test rig

4.3 Pavement locations for field study

The pavement locations used for our field study are described in table 4.1. The photographs of each site are either taken from current NZTA high-speed data records, or are photographs taken during our investigation works for this project.

Table 4.1 Pavement locations for field study

Site	SH	RS	Start	End	Length	Lanes	Treatment type	Treatment date	Site photograph
1	1	664	7.750	7.950	0.200	2	FBS	2007	
2	1	680	8.700	9.350	0.650	2	FBS	2007	

Site	SH	RS	Start	End	Length	Lanes	Treatment type	Treatment date	Site photograph
3	5	99	7.310	9.580	2.270	2	Cement	2008	
4	5	150	17.300	17.600	0.300	2	Cement	2000	
5	5	150	19.680	20.000	0.320	2	FBS	2008	
	5	169	0.000	0.050	0.050	2			
6	5	169	7.635	8.260	0.625	2	Cement	2009	
7	5	190	0.700	1.375	0.675	2	Cement	2010	
8	5	204	6.500	7.200	0.700	2	Cement	2005	

Site	SH	RS	Start	End	Length	Lanes	Treatment type	Treatment date	Site photograph
9	2	592	14.200	14.450	0.250	2	Cement	2003	
10	2	608	5.420	6.410	0.990	2	Cement	2002	
11	2	608	15.730	17.130	1.400	2	Cement	1996 & 1998	
12	2	678	10.240	11.680	1.440	2	Cement	2008	
13	2	678	11.680	12.550	0.870	2	Cement-bound CTB	1996	

Site	SH	RS	Start	End	Length	Lanes	Treatment type	Treatment date	Site photograph
14	2	691	10.580	11.340	0.760	2	Cement	1997	
15	2	721	7.495	7.845	0.350	2	Cement	1997	
16	Urban Gisborne	N/A	0.0	0.340	0.340	2	Cement	2008	
17	2	562	12.66	13.83	1.17	2	Cement	2000	

4.4 Field test-site treatment information

We used data obtained from the various pavement ‘owners’ to assemble the information about each site included in our field test programme (see table 4.2).

Table 4.2 Treatment information for pavement sites included in field testing

Site	Treatment	Treatment date	Treatment description and current condition
1	Foamed bitumen	2007	Existing pavement on SH 1 made up with ‘dacite’ granular overlay in 2006. Addition of new granular make-up aggregate followed by foamed bitumen/cement modification undertaken in winter 2007 to address rutting/cracking. Current condition: Good, with no obvious defects requiring intervention.
2	Foamed bitumen	2007	Existing pavement on SH 1 made up with ‘dacite’ granular overlay in 2006. Addition of new granular make-up aggregate followed by foamed bitumen/cement modification undertaken in winter 2007 to address rutting/cracking. Current condition: Premature seal failure occurred. Seal removed and resealed. Apart from some residual surface roughness/SCRIM issues, no obvious defects requiring intervention.
3	Cement	2008	Existing pavement on SH 5 overlaid with cement-modified ‘dacite’ granular layer as a ‘trial pavement’. Current condition: Good, with no obvious defects requiring intervention.
4	Cement	2000	Top cement-stabilised ‘dacite’ – 3% cement 200mm, over 300mm pavement, volcanic subgrade – in response to premature rutting/cracking in conventional overlay. Current condition: Good, with no obvious defects requiring intervention.
5	Foamed bitumen	2008	Existing pavement on SH 5 overlaid with foamed bitumen/cement-modified ‘dacite’ granular layer as a ‘trial pavement’. Current condition: Good, with no obvious defects requiring intervention.
6	Cement	2009	AWPT (area-wide pavement treatment) on SH 5 involving granular overlay with make-up aggregate, followed by in-situ cement modification to +200mm deep above lively/wet volcanic subgrade. Current condition: Premature rutting/cracking occurred soon after construction. This then settled down over summer. Ruts now ‘wheeltrack sealed’ only, followed by site reseal in March 2011. Pavement condition now appears stable and sound.
7	Cement	2010	AWPT treatment on SH 5 for SCRIM, 100mm overlay, followed by top 250mm stabilised with 3% cement on overall pavement of 400mm. Current condition: Good, with no obvious defects requiring intervention.
8	Cement	2005	AWPT treatment on SH 5, no overlay, existing pavement stabilised to 250mm depth with 3% cement. Original granular pavement depth was 250–330mm. Pumiceous SILT subgrade from depth of 460mm. Current condition: Early premature cracking generally controlled by reseals. Some cracking/pavement repairs ongoing. Surface roughness an issue.

Site	Treatment	Treatment date	Treatment description and current condition
9	Cement	2003	AWPT treatment on SH 2, 100mm overlay, followed by top 250mm depth stabilised with 3% cement. Overall pavement depth uncertain. Current condition: Early premature cracking generally held by reseals until early 2011, when cracking and near-surface 'shoving' accelerated, requiring intervention, notably in the south-bound, increasing RP lane.
10	Cement	2002	Long hill country site, AWPT treatment on SH 2, top overlaid with 50mm, then 250mm 3% cement stabiliser, overall pavement depth 300–350mm, groundwater. Current condition: Early premature cracking generally held by reseals until 2009, when cracking and near-surface 'shoving' accelerated, requiring intervention in localised areas.
11	Cement	1998	AWPT treatments on SH 2, RP 15.46 to 16.05, 200mm stabiliser top 3% cement (1996/96) with overall pavement 400mm, subgrade CBR 5%, from RP 16.05 to 17.28 (1998/99) top cement stabiliser 3% to 200mm, overall pavement 300–500mm on firm subgrade. Current condition: Good, with no obvious defects requiring intervention. First section is the Kareeara Control site (refer to section 4.7.2).
12	Cement	2008	AWPT treatments SH 2 south of Hastings, 125mm overlay, followed by top 250mm depth stabilised with 2% cement. Overall pavement depth 450–500mm. Current condition: Good, with no obvious defects requiring intervention.
13	Cement-bound CTB	1996	Early trial for cement-treated basecourse (CTB) pavement 'Byfords' – 150–200mm of imported premixed cement-bound pavement (>5% cement) over pavement 400mm deep Current condition: Early widely spaced longitudinal and transverse cracking generally held by reseals and crack sealing. Some cracking and near surface "failure" has accelerated requiring intervention in localised areas
14	Cement	1997	AWPT treatment on SH 2, top overlaid with 75mm, then 200mm 3% cement stabilisation, overall pavement 350–580mm deep. Current condition: Reasonable condition, some isolated outer wheeltrack cracking/shoving requiring intervention. One of the early recycling 'success' stories, as previous treatments (eg reseals) had not worked.
15	Cement	1997	Early AWPT/recycle treatment on SH 2 south of Waipukurau, top 200mm 3% cement stabiliser in overall pavement, around 500mm deep. Current condition: Good, with no obvious defects requiring intervention. Initial surface roughness issues following construction had not deteriorated any further.
16	Cement	2008	New granular overlay pavement that failed prematurely. As a remedial work, top surface cement-stabilised with 2% cement to a depth of 200mm. This then failed by cracking/rutting within a year, notably in the heavily loaded traffic lane. Current Condition: Requires urgent repair.
17	Cement	2000	Early AWPT on SH 2, 3% cement stabiliser to 250mm deep. Current condition: Good, with no obvious defects requiring intervention.

4.5 Field testing – pavement cores

Our field testing included the coring and retrieval of samples from each of the modified pavement layers at 17 sites. Dry, compressed-air-supported core retrieval was successful on all sites except site 13. In this latter case, the firmer cement-bound layer (CTB) needed drilling with water assistance.

The fact that cores were able to be retrieved from all these sites is characteristic of more than 'modified' behaviour. Whilst in theory, modified pavement materials should behave as an unbound material, our field testing showed that the addition of even small quantities of cement consistently produces an in-situ material that behaves like it is lightly bound. We expect the cement reaction is also influenced by the mineralogy[4] of the aggregate materials.

Figure 4.3 shows three representative pavement-layer cores. In all cases, the cores retained their shape without assistance. During retrieval the cores appeared to come loose at or near to the separation line between modified and unbound materials, except where existing cracks or infiltration (of water) had created a zone of weakness. Existing seal layers are at the top of each core shown.

Figure 4.3 Photographs of representative pavement cores retrieved from the field

4 Allan Tuck, Higgins Contractors Ltd (HB), pers comm

4.6 Laboratory testing

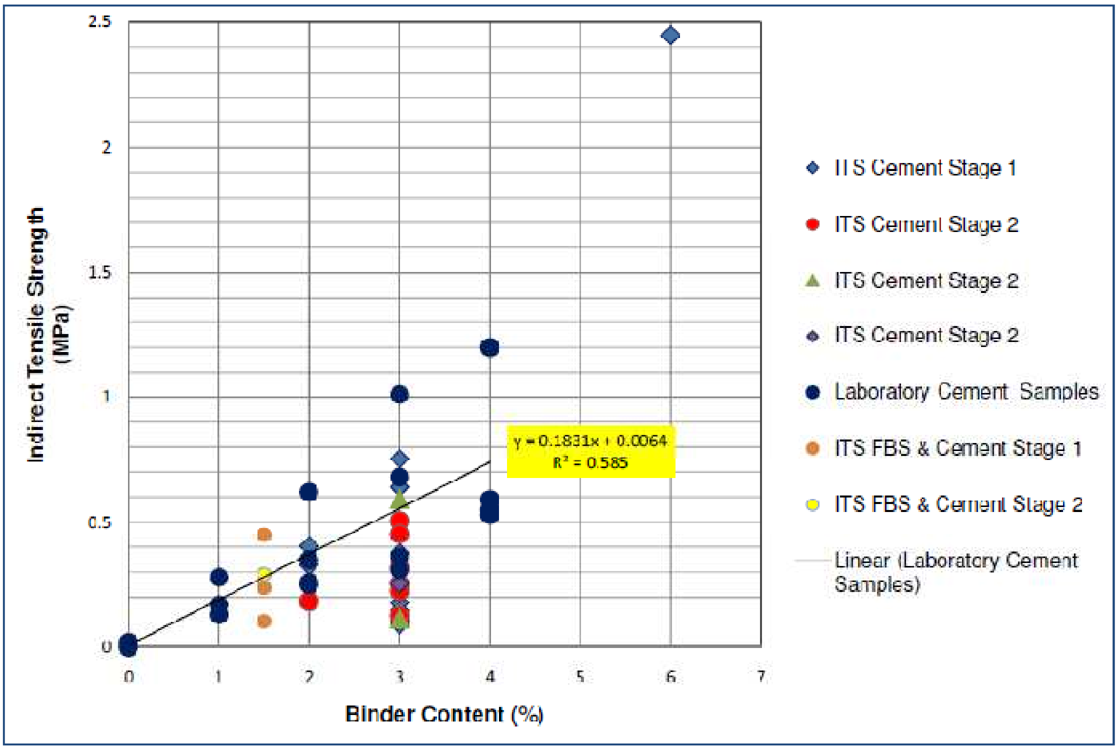
The cores we retrieved from the field sampling were returned to the Opus Napier Engineering Laboratory. Here they were logged and photographed. This complete data set can be found in appendix B: Laboratory and field test results.

The core samples were tested in the following two stages:

1. ITS[5] tests were completed in Napier on at least one intact sample from each site. (We chose to use the ITS test because of the often irregular as-sampled dimensions.)
2. ITS tests were continued at a later stage in the Auckland laboratory facility. UCS[6] tests were also undertaken, including measurement of elastic modulus (refer back to figure 3.3).

We compared the ITS results from each core and related these to the measured – or in some cases, with old sites, assumed binder contents (mass of binder/mass of aggregate). The results are shown in figure 4.4.

Figure 4.4 Comparison between ITS and binder content from core samples from laboratory testing



As expected, the reported ITS increased with increasing binder content. The spread of ITS results, at binder content of 3% in particular, was large.

The spread of the UCS (and to a lesser extent the ITS) data in our data inventory was reasonably large. The spread of the data could possibly be explained by:

- changes in compaction efforts for the prepared samples

5 NZS 3112, Part 2, 1986: test 8

6 NZS 4402 1986: test 6.2.1

- changes in curing conditions

Whilst the spread of UCS data is reasonably large (see figure 3.3), it is clear that in the majority of cases our reported UCS results exceed the expected range of UCS for modified materials of 0.7–1.5MPa.

The laboratory cement samples reported in figure 4.4 covered a range of binder content up to 4%. These were prepared as part of an investigation programme for ongoing recycling work in Hawke's Bay. The aggregate materials are typically greywacke in origin. The samples were prepared using heavy compaction. We plotted a linear regression line to this data, as shown in figure 4.4. The data fit for the laboratory cement-only samples was relatively poor (R^2 = 0.585), but was the best of possible linear regression outcomes.

The three results listed at a binder content of 1.5% show the relative position of the FBS sites, when plotted at the cement content only (assumed to be 1.5%). It appears, as expected, that the cement content in the FBS cores governs the ITS result.

Modified materials incorporating granular materials and reactive agents such as cement are characterised as having ITS strengths of equal to, or less than, 80kPa (refer to section 1.3, table 1.1). Based on this premise, the data shown in figure 4.4 would appear to suggest that 'modified' behaviour is characterised by cement-binder contents $\leq$1%. As noted above, it may also be influenced by aggregate mineralogy.

The foamed bitumen outcomes in figure 4.4 appear to be consistent with modified pavement behaviour. We understand it is a fundamental philosophy of foamed bitumen that the material behaves as modified – the failure mode is ductile rather than brittle/cracking. Whilst it is difficult to consider comparing the blended binder with cement-only binder ITS data as in figure 4.4 (because of the potentially different performance mechanisms: brittle vs ductile), the cement content in the FBS cores does appear to govern the ITS result.

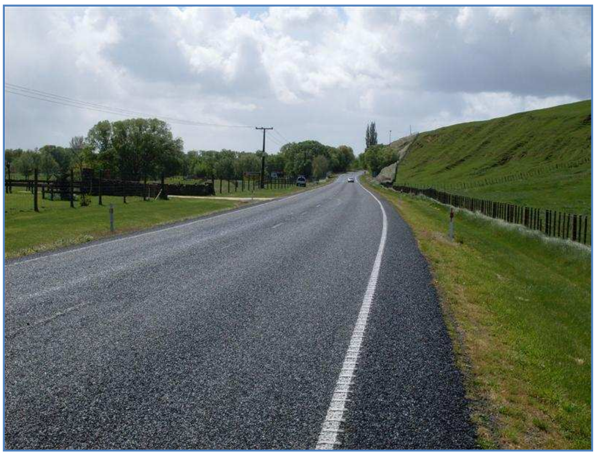
4.7 Case study project sites

One of the objectives of this research project was to improve our understanding of the changes that occur in cement-modified pavement layers. From the investigation and performance data we received, we chose five projects to act as 'performance markers'. At the time of writing, three of these projects were continuing to perform well. The other two projects had required premature and unplanned maintenance intervention. These projects are described below.

4.7.1 Pukeora project: SH 2, RP 721, 7.496 to 7.845 – constructed 1997/98 (site 15 in table 4.1)

The Pukeora project was on SH 2 south of Waipukurau, on a relatively low-lying flat site (see figure 4.5) The original pavement consisted of 50–60mm of chipseal (multiple layers) overlying partially crushed granular pavement, giving an overall depth of approximately 500mm. AWPT was programmed to address SCRIM and top-surface-layer maintenance defects. The subgrade was gravelly SILT. The existing near-surface seal and pavement was stabilised to a depth of 200mm with 3% cement by dry weight. Following several reseals since construction, at the time of this research the existing pavement was slightly rough (this was also the case immediately following construction), and had required no unplanned intervention.

Figure 4.5 The Pukeora project



4.7.2 Kareeara project: SH 2, RP 608, 15.46 to 16.05 – constructed 1995/96 (part of site 11 in table 4.1)

The Kareeara project was a hill-country site north of Napier (on the route to Wairoa/Gisborne), within a narrow valley. The original pavement had 45–75mm of chipseal, with the overall pavement structure being variable gravels of between 230mm and 270mm, in some places overlying sub-base gravels, including limestone to a total depth of approximately 400mm. A subgrade CBR of 5% was indicated by FWD. The underlying subgrade is sandstone. The adjoining water tables were wet. AWPT was programmed to address SCRIM and top-surface-layer maintenance defects.

The existing near-surface seal and pavement was stabilised to a depth of 200mm with 3% cement by dry weight. Following several reseals since construction, the pavement had had no unplanned maintenance.

Figure 4.6 The Kareeara project

4.7.3 Territorials project: SH 2, RP 592, 14.2 to 14.45 – constructed 2002/2003 (site 9 in table 4.1)

The Territorials project was a 'truck-ride' treatment site on rolling hill country north of Lake Tutira, on the route to Wairoa from Napier. The original pavement composition was not investigated. The existing pavement was overlaid with 100mm of crushed aggregate (AP40) and then the combined upper-pavement layers were stabilised to a depth of 250mm with 3% cement by dry weight.

Soon after construction, widely spaced transverse and longitudinal cracks were cracked-sealed. After that, the pavement remained reasonably stable (including reseals) until early 2010, when wheeltrack cracking, shallow shear/shoving occurred, notably along the outer wheeltrack. Whilst lateral creep within the underlying substrate could have contributed to the cracking, lateral shear is the most likely cause.

Figure 4.7 The Territorials project

4.7.4 Awaho Culvert project: SH 2, RP 544, 3.56 to 4.4 – constructed 1997/98

The Awaho Culvert project was a low-lying, flat valley just south of Wairoa on SH 2. The original pavement consisted of 40–60mm of chipseal overlying 320–380mm of variable gravels. The subgrade, alluvial SILT with CBR of 3–5%, had a higher winter water table. The existing near-surface seal and pavement was stabilised to a depth of 200mm with 3% cement by dry weight, from RP 3.56 to 4.00. From RP 4.00 to 4.4, the existing pavement was first overlaid with 100mm of crushed aggregate (AP40.

The pavement had been resealed several times since construction. At the time of this research, minor wheeltrack rutting and pavement repairs suggested that the stabilised pavement layer was reverting back to an unbound state.

Figure 4.8 The Awaho Culvert project

4.7.5 Hirini Street, Gisborne – constructed 2008 to 2010 (site 16 in table 4.1)

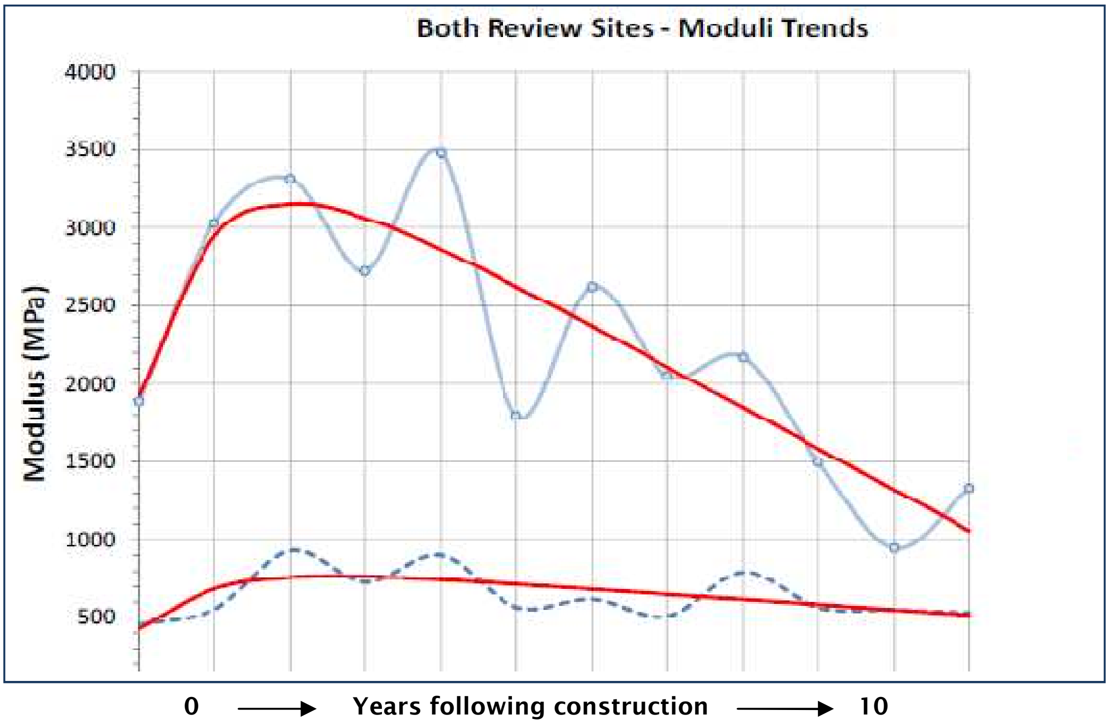
Hirini Street was first constructed in 2008 using unbound granular pavement (M/4 basecourse overlying sub-base). Within the first year, rutting in the basecourse on the more heavily trafficked port inbound lane was treated, using stabilisation with 2% cement by dry weight to 200mm depth. Then again in 2010, rutting and cracking in the cement-stabilised upper pavement prompted further investigation. The more heavily trafficked inbound lane had much more distress, Traffic loading records for that lane were then used to estimate 'loading to failure'.

Figure 4.9 The Hirini Street project

4.7.6 Historic review of the Awaho and Kareeara project sites

As part of their annual plan review since 1997, the NZTA has commissioned Opus and Tonkin & Taylor, to complete annual assessments of the effectiveness of recycling projects, including the Awaho and Kareeara projects. In figure 4.10 we show the combined 10th and 50th percentile (hatched blue and solid blue line respectively) back-calculated top-layer modulus from both sites (plotted on a linear scale in this instance), to provide a general indication of the change in post-construction back-calculated stabilised-layer modulus. Unfortunately the 90th percentile data was not consistent enough to provide a stable trend line.

Figure 4.10 Time-related changes in stabilised-layer modulus



Trend lines (▬▬) were fitted to this data, based on our expectation that the stabilised-layer modulus should increase initially and then decrease over time with the wear and degradation induced by ongoing trafficking; therefore, convex upward trend lines were subjectively fitted to the data collected so far.

The changes in modulus shown in the above figure appear to be converging at some time after 10 years. The 10th percentile modulus appears to be relatively consistent over time (between 750 and 500MPa over an 11-year period). From earlier highs, the 50th percentile modulus is decreasing and it appears this may converge with the 10th percentile modulus at around 500MPa in a few years' time, at which point the pavement will have reverted back to the unbound condition. This will not automatically mean that maintenance intervention will be required. In fact, our experience shows that these pavements can continue to perform well.

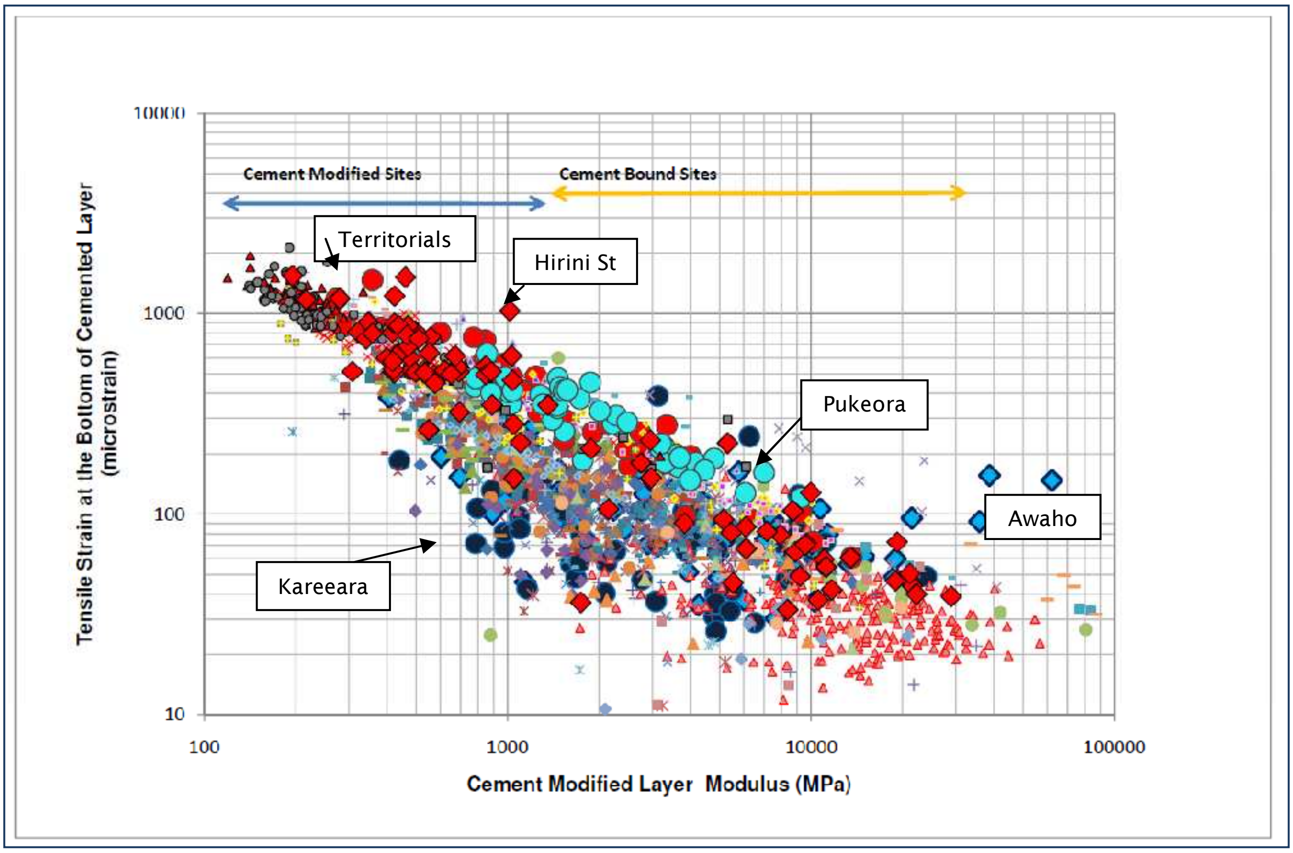
Our observation of these recycling projects suggests that this 'change in life' from lightly bound to unbound does not necessarily result in dramatic failure requiring expensive maintenance intervention, provided that ongoing reseals have maintained pavement waterproofing and the underlying pavement is sound. We surmise that provided the underlying pavement structure is compatible with a 'stable' recycled layer, over time (in this case around 10 years) the 'stiffer' portions revert back to a high-quality granular equivalent, while the residual 500MPa sections within or around the recycled material 'live there quite nicely'.

4.7.7 Comparison of pavement performance

We used the data collected in the pavement inventory (refer to section 3.1) to compare the performance of the historic pavement sections. In particular, we investigated how the five case study sites described above could be characterised within the wider inventory data.

In figure 4.11, the back-calculated performance outcomes of the five case study project sites (see sections 4.7.1 to 4.7.5) are compared to the back-analysis outcomes derived from the wider inventory data. The two sites where premature failure had occurred are represented by large red markers, while the three sites that had not, at the time of this research, required significant maintenance intervention are shown with large dark- and light-blue markers.

Figure 4.11 Performance outcomes from various cement-stabilised projects



We then investigated whether the results shown in the above figure could be used to develop a model of the expected pavement life. We recognised that the latter would be affected by design traffic loading, pavement layer modulus, and layer depth and subgrade condition. It was interesting that the stabilised-layer moduli in the three older pavements (Kareeara, Awaho and Pukeora) were still positioned within the lightly bound category as currently prescribed by Austroads. Hirini St was the youngest project where the pavement failures were dramatic, but still largely localised to the more heavily loaded lane. This could explain the spread of data points for Hirini St in the above figure.

5 Quantifying pavement life for cement-modified pavement layers

5.1 Introduction

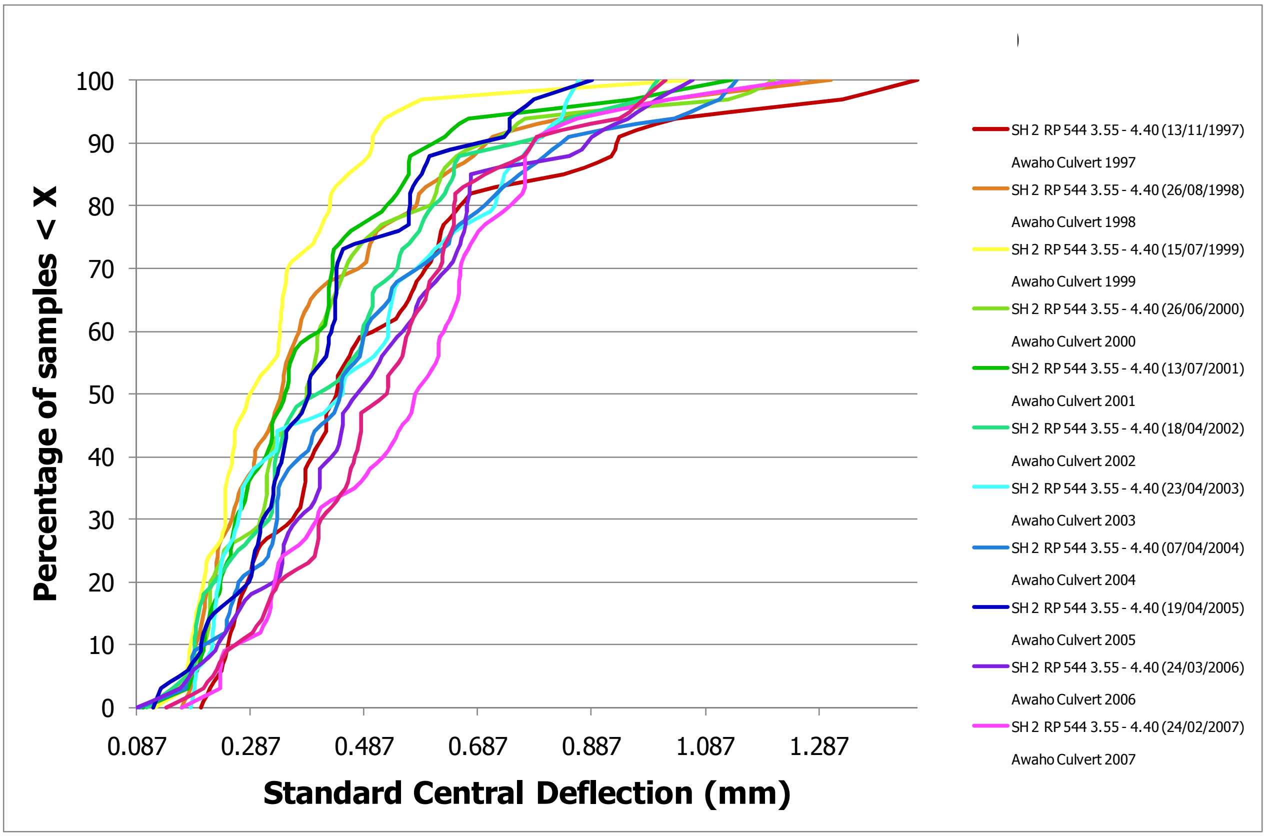
The shape of the performance data presented in figure 4.11 in the previous section prompted us to investigate whether we could quantify the 'design life' for pavements with cement-modified upper pavement layers. In this context, and based on the findings of our field investigations, stabilised materials (those stabilised with cement and/or cement with another additive) are initially lightly bound materials.

Our research of this topic was undertaken in five phases, as outlined in the following sections.

5.2 Phase 1: Pavement life, central deflection (d0) and curvature (d0-d200)

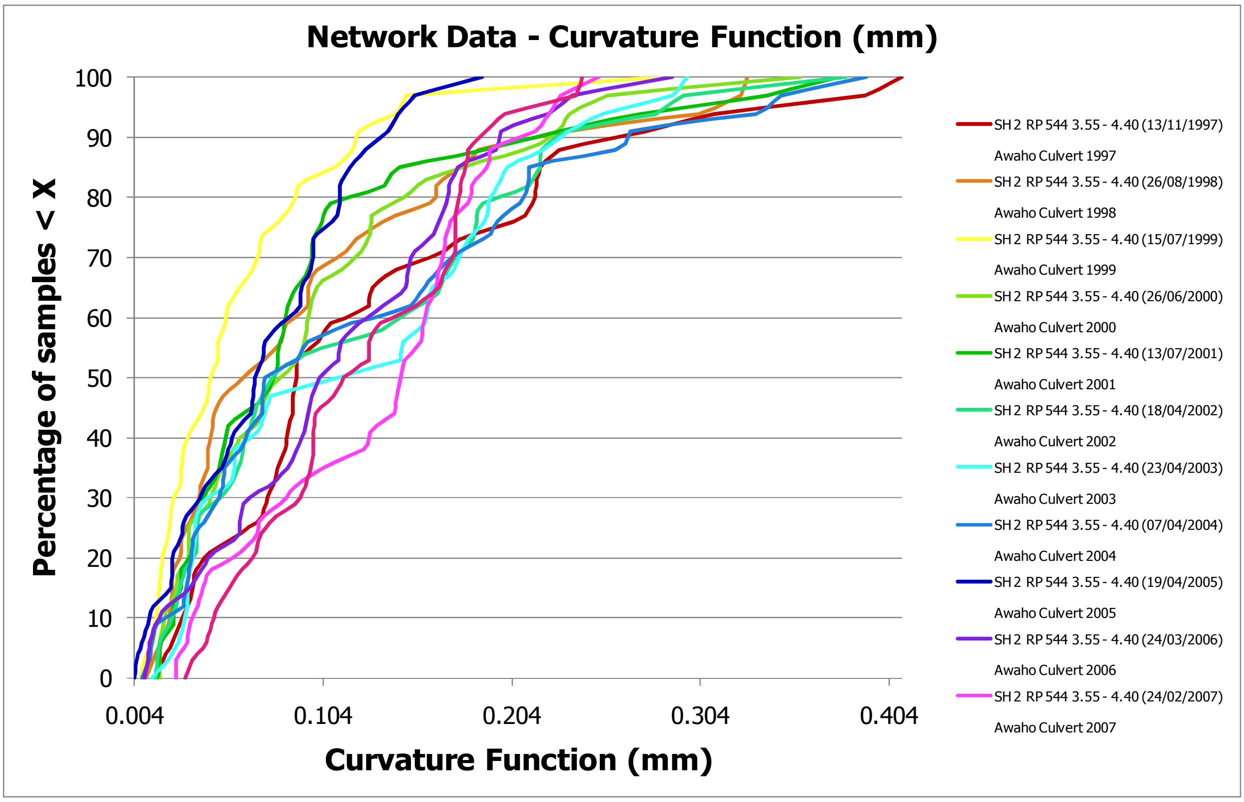
In the first phase, we investigated whether a relationship could be found between expected pavement life and surface deflection and curvature (d0 and (d0-d200)). We used 10 years of FWD pavement performance data (1997–2007) from the Awaho Culvert site (rehabilitation date 1997/98) to investigate the change in central deflection (figure 5.1) and curvature (figure 5.2). The red line in both graphs represents the pre-treatment condition and the yellow line the immediate post-construction outcomes. The most recent data (2007) is coloured mauve.

Figure 5.1 Central deflection data from Awaho Culvert project



As expected, the central deflections reduced soon after construction as the treated basecourse cured and stiffened (the yellow trace in the above figure). It increased thereafter over time, albeit in an irregular pattern as the stabilised aggregate became less bound and the underlying pavement materials exerted greater influence on deflection. In this case, after 10 years the 10th percentile and 50th percentile deflections (d0) were higher than the pre-treatment values. The 90th percentile values were less conclusive, but did confirm an increase in deflection over time following treatment. A similar behaviour is seen in the curvature data (figure 5.2).

Figure 5.2 Curvature data from Awaho Culvert site



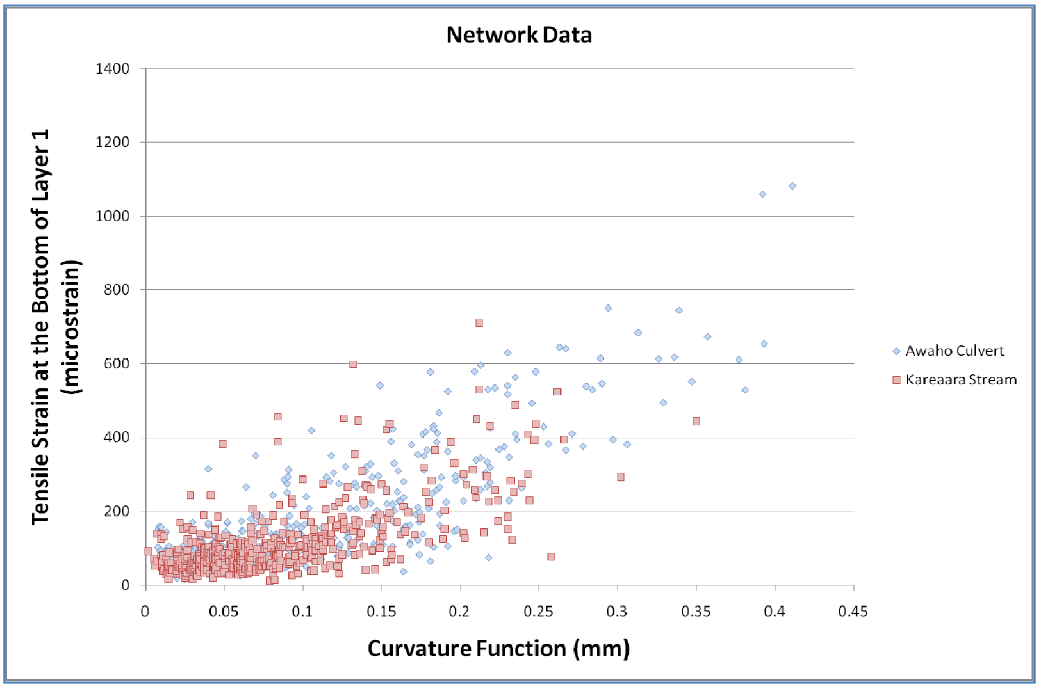
We investigated whether a relationship could be found between the back-calculated tensile strain in the cement-stabilised layer and curvature, using data from the Awaho and Kareeara sites (see figure 5.3).

We know that the d0, curvature (d0-d200) and tensile strain at the base of the stabilised basecourse layer are a function of the loading condition, top-layer modulus, the stiffness of the underlying unbound layer or layers, and (indirectly) of the subgrade. The stiffness (modulus) of the underlying layers (not the subgrade) is not usually derived directly from d0 or curvature alone.

The use of d0, curvature (d0-d200) or other simple bowl parameters to predict pavement performance would certainly be desirable. We would be able to use readily available information (without additional interpretation). A multitude of other bowl parameters (combination of deflections from various geophones in the FWD dataset) were trialled in an attempt to make sense of the spread of data in figures 5.1–5.3, but unfortunately our results were inconclusive.

At this stage of the study we concluded that the two parameters used by Austroads to model fatigue of bound materials (modulus of the stabilised layer and horizontal tensile strain) provided a better starting point for a New Zealand model than any of the more fundamental bowl parameters.

Figure 5.3 Comparison between tensile strain and curvature function



5.3 Phase 2: Pavement life and Austroads fatigue equations

Fatigue relationships for cemented materials are based on the Austroads general equation 6.4 (Austroads 2008) as shown in the next figure.

Figure 5.4 Background to cement-bound fatigue equation

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Fatigue criteria

Fatigue relationships have been derived for cemented materials having various modulus values and they may be used to give an indication of fatigue life. The relationships have been derived from overseas research work and may be used where more reliable information is unavailable.

Limited information is available on the in-service fatigue of cemented materials used in Australia, except for several Accelerated Loading Facility (ALF) trials (e.g. Jameson et al. 1992b, 1995 and 1996; Vuong et al. 1996).

These relationships given in the following general equation, and shown in Figure 6.5, generally have been found to be in accordance with observed performance:

$$N = RF\left[\frac{\left(113000/E^{0.804} + 191\right)}{\mu\varepsilon}\right]^{12} \qquad 6.4$$

where:

N = allowable number of repetitions of the load

$\mu\varepsilon$ = tensile strain produced by the load (microstrain)

E = cemented material modulus (MPa)

RF = reliability factor for cemented materials fatigue (Table 6.8)

These relationships utilise reliability factors (effectively a 'shift factor') – see Austroads table 6.8 in the next figure.

Figure 5.5 Reliability factors used with cement-bound fatigue equation

Table 6.8: Suggested reliability factors (RF) for cemented materials fatigue

Desired project reliability				
80%	85%	90%	95%	97.5%
4.7	3.3	2.0	1.0	0.5

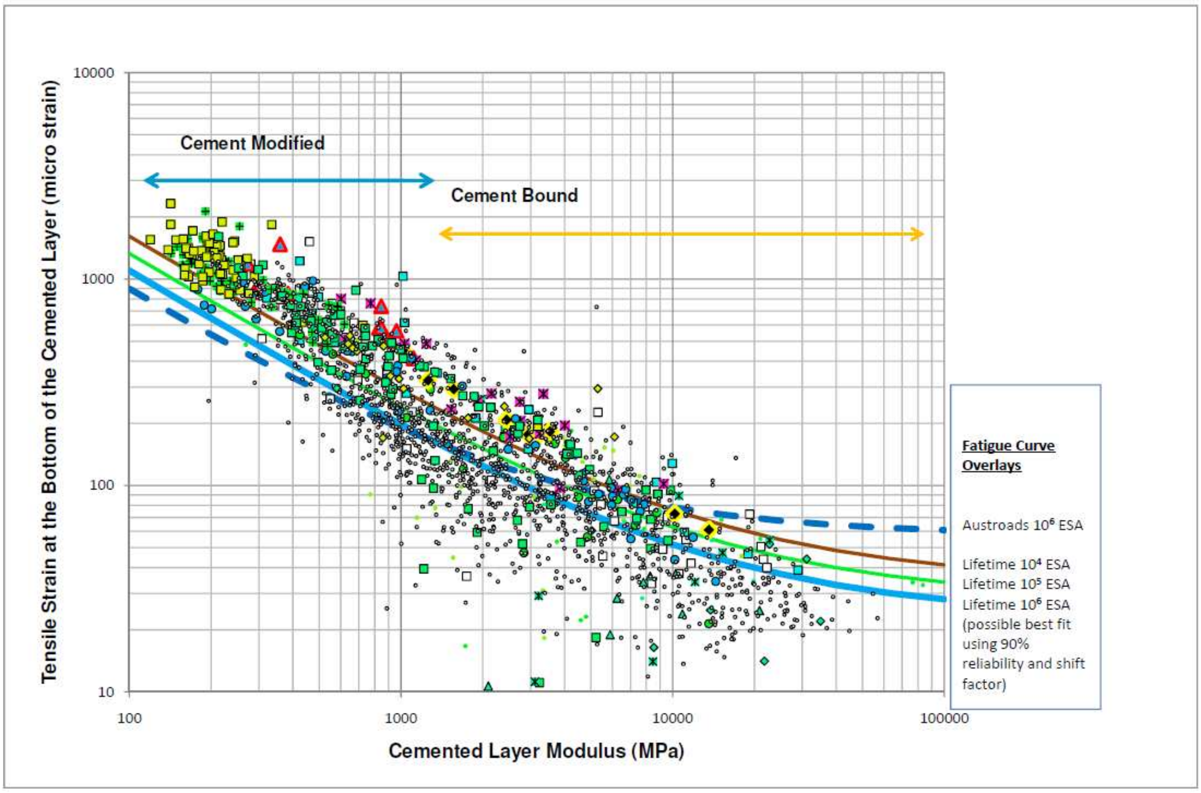
Equation 6.4 in Austroads 2008 has been rewritten as:

$$E = f(\mu\varepsilon)$$

$E = f(\mu\varepsilon)$ (for constant N and RF) (Equation 5.1)

To further develop the idea of fatigue criteria for lightly bound pavement layers, we plotted data from the rearranged equation above onto a graph of $\mu\varepsilon$ versus E for fixed values of N ranging from 0.01–10 MESA[7]. We then compared this with the plots of the measured performance data from this project's FWD tests. We fitted 'lifetime' performance curves based on the Austroads model to the upper 10th percentile of tensile-strain data obtained from sites where 'failures' were known to have occurred, by adjusting the shift factor (SF) to best replicate actual performance. Trials with various values of SF suggested that an SF of 15 was appropriate for the limited New Zealand data available so far, as shown in figure 5.6.

Figure 5.6 Quantifying cement-stabilised pavement life (PR=90%, SAR12/ESA[8]=12, SF = 15)



7 MESA – million ESA

8 ESA – equivalent standard axle

The above figure also displays the current Austroads performance curve for 10^6ESA (dotted blue line).

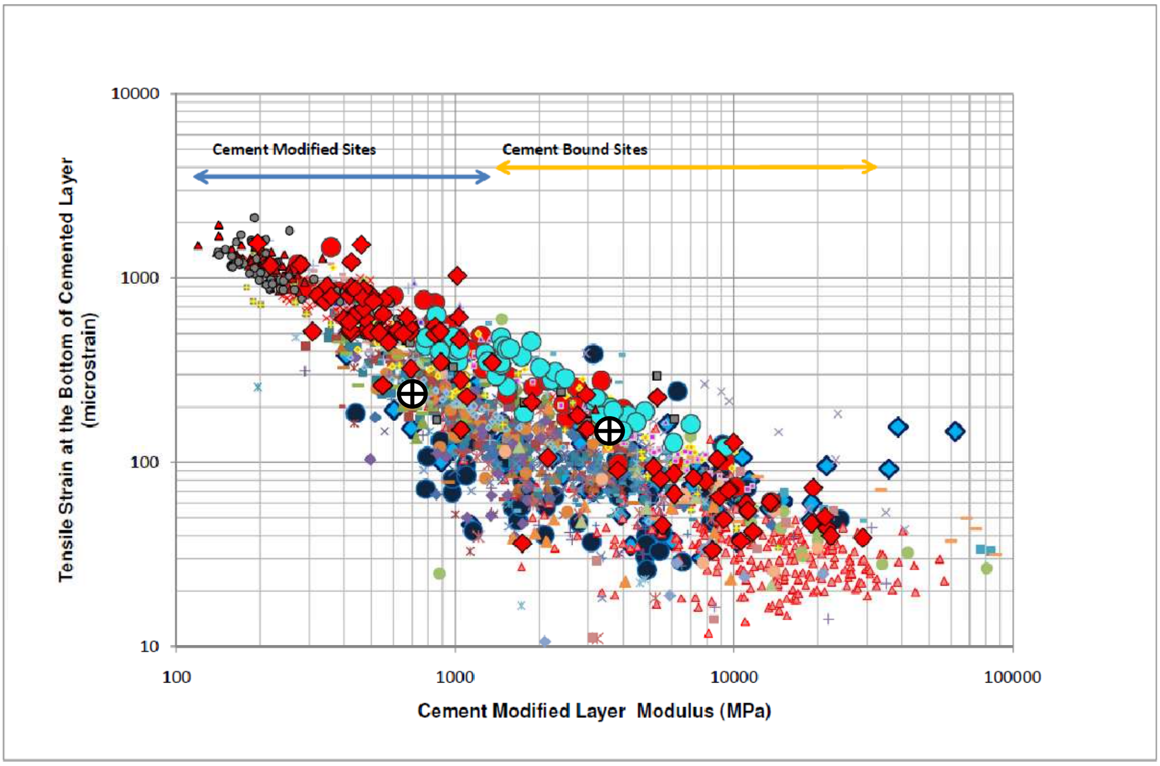
To test this approach against a sample of known performance data, we used Austroads equation 6.4 to derive fatigue relationships for layer moduli of 700MPa and 3500MPa respectively, as shown below:

$$N\ (700\ \ MPa\ \ ) = \left[\frac{774}{\mu\varepsilon}\right]^{12} \qquad N\ (3500\ \ MPa\ \ ) = \left[\frac{350}{\mu\varepsilon}\right]^{12}$$

(Equation 5.2)

Then for a design traffic loading of 1MESA (10^6ESA), these relationships provide for a maximum allowable tensile strain at the base of the cement-stabilised layer of approximately 245µε and 111µε respectively. These results are shown plotted as dark black markers (⊕) in figure 5.7. The light blue markers ( ) are indicators for one of the oldest recycling sites in Hawke's Bay, which at the time of this writing, was still performing well. We were encouraged that for the cement modulus range between 700MPa and 3500MPa, the vast majority of the large red markers (indicating premature failures in our control studies) lie above these black markers.

Figure 5.7 Comparison of stabilised-layer tensile-strain data with the Austroads fatigue equation – cement-modified layer modulus of 700MPa and 3500MPa, N = 1MESA



The presentation in the above figure initially created some confusion for the research team. It *appeared* to suggest that the existing Austroads fatigue equations for the selected layer modulus of 700MPa and 3500MPa could be aligned quite well to our performance data. However, this was not our experience. Many of the project sites we studied had outlived, by a significant margin, the life predictions delivered by the Austroads equation 6.4. For example, the Awaho and Kareeara sites had not required any significant maintenance intervention, nor shown any evidence of cracking, after more than 10 years, which is more than 10 times the predicted fatigue life for these pavements.

We were then reminded that the calculations that sit alongside the two 'Austroads' points in the above figure would need to include a factor for project reliability (RF) (95% RF is assumed with factor 1.0) and a presumptive damage index for cemented layer fatigue SAR_c/ESA factor of 3.6 (see figure 5.8).

Figure 5.8 Presumptive damage factors (Transit NZ 2007)

7.4 Pavement Damage in Terms of Standard Axle Repetitions
(refer APDG 7.6.2)

Presumptive Damage Index parameters have been determined from New Zealand WIM data (see Table 7.8).

Table 7.8 Presumptive Damage Index parameters for New Zealand traffic loading conditions.

Damage Type	Unit	Value	Damage Index	Value
Overall Damage	ESA/HVAG	0.6	N/a	N/a
Asphalt Fatigue	SAR_a/HVAG	0.6	SAR_a/ESA	1.0
Subgrade Rutting	SAR_s/HVAG	0.8	SAR_s/ESA	1.2
Cemented Layer Fatigue	SAR_c/HVAG	2.2	SAR_c/ESA	3.6

This would in turn mean that if we were using the two pavement options with layer moduli of 700MPa and 3500MPa respectively, at the strain levels shown in figure 5.7, then the expected design traffic loading would only be 1MESA/3.6 or 278,000 ESA at 95% PR. Obviously the economic viability of the cement-stabilised basecourse solution would come into question if this were the case.

Armed with this knowledge, the research team then sought to step away from seeking the best fit of the performance data using Austroads equation 6.4, and investigated how well the use of this form of equation could predict the observed pavement performance on our five case study projects, and on other project sites in the data base.

Using the equation parameters shown in table 5.1 (which are based on current guidelines), we modelled the predicted pavement life for pavements within our dataset, and compared these to observed pavement performance, as shown in figure 5.9. With these input parameters, the spread of data on the observed-life vs predicted-life curve was quite large (sum of squares >7000 in figure 5.10).

Table 5.1 Parameters used in phase 2 pavement life modelling

PR =	95%	a =	113000
SF =	1	b =	0.804
RF =	1	c =	191
SAR12/ESA =	1	d =	12
		e=	0

Figure 5.9 Phase 2 pavement life modelling outcome

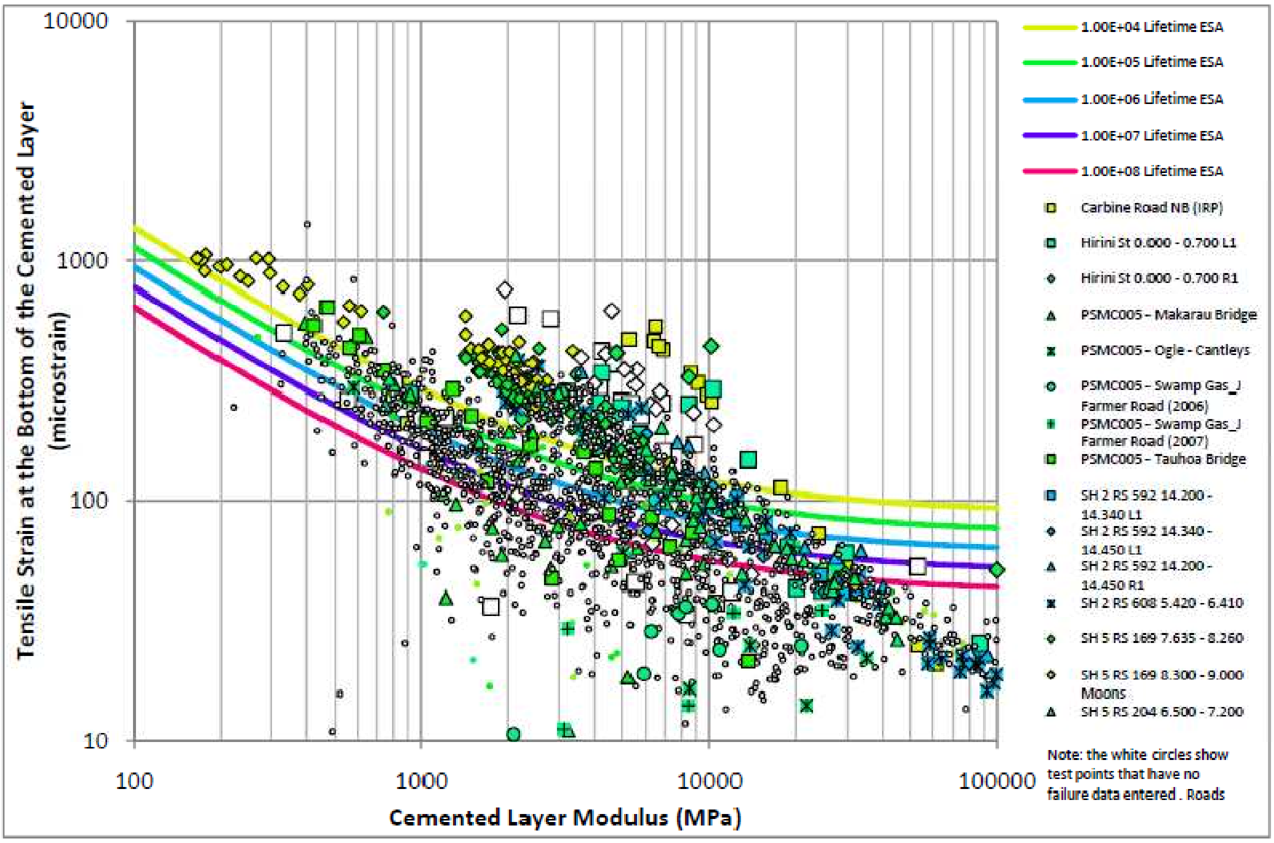
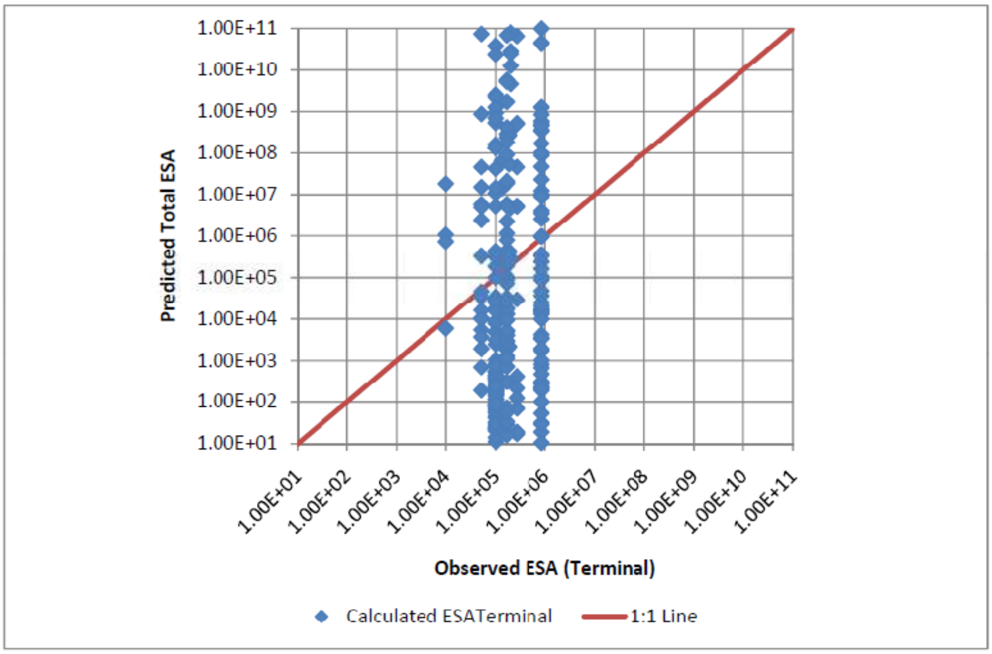


Figure 5.10 Spread of predicted pavement life vs observed pavement life from phase 2 modelling



The research team then set about using ideas to refine the modelling equation to better fit the observed performance data, as discussed in section 5.4.

5.4 Phase 3: Pavement life, modified Austroads fatigue equations, and change in the modulus of the cement-treated layer over time

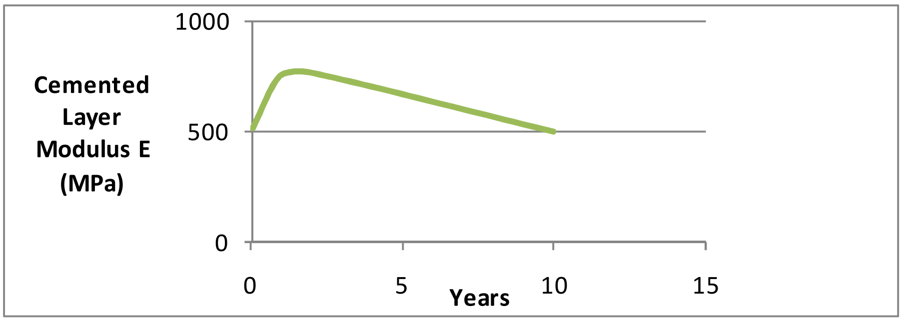
At this point of our research, it became very apparent that we were using in-situ back-calculated pavement layer moduli from sites with different ages (and therefore different stages of curing). The pavement sections were also in differing stages of fatigue. These factors would change the cement-stabilised layer modulus and overall pavement behaviour.

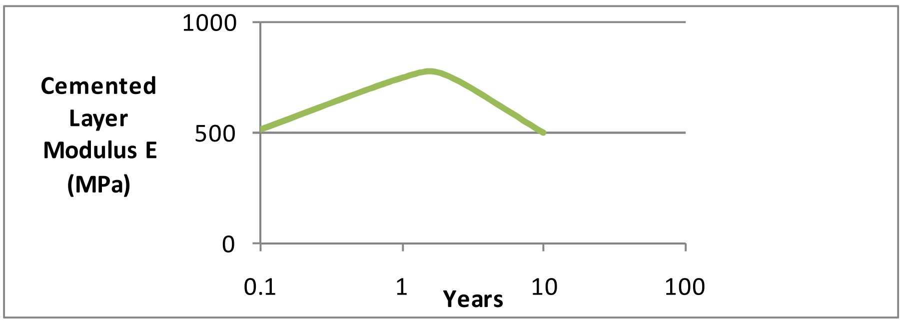
It is known that full cement curing takes 1–2 years (Croney 1998), and a progressive increase in modulus can be expected following construction. The strength gain is asymptotic in nature, with the strength-gain curve flattening with time. Hence in order to develop a fatigue relationship, we resolved that it would be necessary to establish a 'reference state' for a modulus of any cement-treated material. This approach has a parallel in asphalt layer modelling, where moduli vary substantially (by an order of magnitude) with temperature. It is established practice worldwide (including Austroads GMP) that in-situ back-calculated moduli for asphalt layers are referenced (corrected to) the weighted mean annual pavement temperature (WMAPT).

The research team investigated how this concept could be used for cement-stabilised materials. We noted that the typical concrete-curing curve reported by Croney (1998) supported the adoption of a cement-treated layer modulus with a standard reference age of one year, denoting this value as E_c.

We then used the performance data obtained from the Awaho project site to investigate how the layer modulus for the cement-stabilised basecourse layer at this site had changed over time. In figure 5.11 we report the change in 10th percentile cemented-layer modulus over the life of the pavement, based on the FWD back-analysis, using a linear scale and then a log scale for the time in years.

Figure 5.11 Awaho Culvert project – change in 10th percentile modulus after 0.6MESA (10 years)

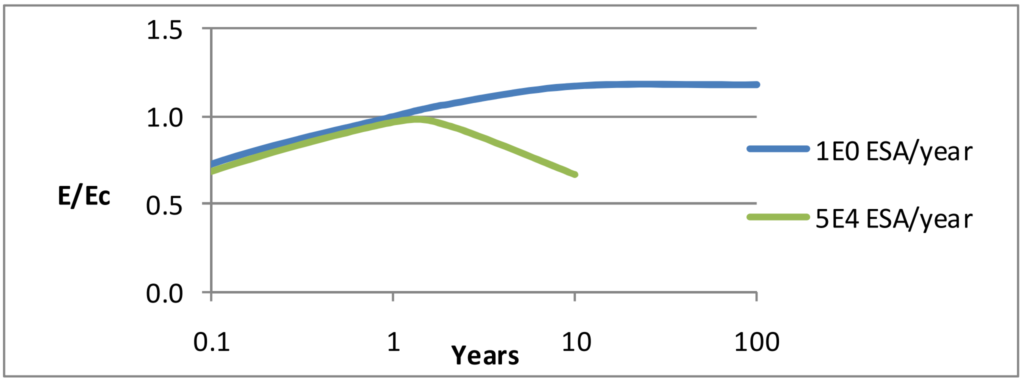




At the Awaho site, after initial stiffening in the first year or so, probably due to ongoing curing (shown more clearly in the above figure), the 10th percentile modulus appears to reduce progressively (presumably due to fatigue micro-cracking with trafficking) at a rate of about 200kPa per million ESA.

The modulus change in the cement-curing curve, with little or no trafficking (shown as the blue line in figure 5.12), and the cement-stabilised layer-curing curve in the active traffic lane (the green line in figure 5.12) can be normalised with respect to a reference modulus. The normalisation in this instance shifts the graphs to pass through a value of 1.0 after 12 months of curing time (see figure 5.12).

Figure 5.12 Normalised change in cemented pavement layer modulus



The reference modulus E_c can then be incorporated into a conceptual pavement-layer performance model as described in section 5.5.

The second correction, for reduction of modulus as tensile fatigue begins to induce cracking, was addressed in our interim model using the data from the Awaho project site to investigate a feasible correction as a function of trafficking (ESA).

5.5 Phase 4: Conceptual model for performance of cemented pavement layer

At this stage, the research team found it necessary to deviate in some aspects from the published Austroads cement-bound fatigue model in order to fit a conceptual pavement performance model to the recorded data for what we now knew were at least initially lightly bound materials.

The underlying principles we retained when developing a conceptual pavement model were as follows:

- The fatigue (cracking) lives of aggregates with binders (cement and/or bitumen) appear to be related to horizontal tensile strains at the bottom of the stabilised layer.
- For a given additive mix, the ratio of the tensile strain in the layer to the tensile strain at failure is likely to be a key parameter governing the pavement’s life.
- The maximum tensile stress in the layer at failure is also expected to be related to the tensile strength of the stabilised material.
- For different additive mixes, the tensile strain at failure is considered (including by Austroads) to be inversely related to modulus.
- Modulus can be measured in the laboratory and in the field, and hence is a parameter more easily and dependably determined than tensile strain at failure.

- In view of the above, a fatigue model that is a function of both tensile strain at the bottom of the stabilised layer, and the stabilised layer modulus, is expected to be the most practical choice.
- Because modulus is very dependent on curing and confinement/support, an in-situ determination is desirable, rather than sole reliance on laboratory testing.
- Modulus of an unbound basecourse remains relatively constant for much of its mature life (provided the basecourse material retains stable material properties), but it appears that stabilised basecourse materials with binders cannot be modelled as simply.
- In the absence of trafficking, the modulus of cement-treated materials increases steadily for the first 1–2 years as the curing process takes place. Foamed-bitumen-treated materials will also be subject to some degree of curing, due primarily to the cement additive.
- After the initial set is achieved, trafficking (in terms of ESA) in the first 1–2 years, N_c, will be accompanied by some degree of self-healing, provided that tensile strength is not exceeded under single wheel loads.
- After the curing is essentially complete (1–2 years), fatigue trafficking (in terms of ESA), N_f, causes progressive reduction in modulus or decay for cement-stabilised sites (eg Awaho and Kareeara sites), provided again that the tensile strength is not exceeded. For this reason, the same fatigue model cannot be used in the curing period as in the post-curing phase – it is a 2-phase model, but is not the same as the 2-phase (pre- and post-cracked) model that is currently used in Austroads. For foamed bitumen, we expect that aging (increase of modulus) will also require modelling.
- For cement-stabilised material, Austroads uses a power law of 12, which provides a model that is very sensitive to the magnitude of the applied load. For low traffic-volume pavements, when the design tensile strain approaches the value for failure (cracking) of the bound layer, high sensitivity is very appropriate. For large traffic-volume pavements, when the material is further away from its measured tensile capacity, a lower power law could be expected. The research team resolved here that a bound-layer model should not be constrained to a constant exponent; rather, the model should explore the use of exponents inversely related to design ESA (ie less constrained than Austroads). This could address the problem with the current Austroads cement-fatigue equation for stabilised materials, which generates a significant number of points as having cracking lives of well over 10^9ESA. For foamed bitumen, a lower power law would be expected for tensile fatigue, similar to that for other bituminous materials.
- For lightly bound material, because the layer modulus increases for the first year or so and then decays with trafficking, in order to compare different materials at different ages and sustained traffic, a 'reference modulus' needs to be nominated. The reference modulus adopted in this study was the in-situ modulus (E_c) of a sample that had had little or no trafficking (eg just outside the wheeltrack), with testing carried out after one year (after the majority of curing was assumed to be complete). Where this information has not been collected, an iterative procedure can be used to back-calculate the reference modulus, using figure 5.12. This is very approximate, but we hope it can be refined in future when more performance data and ongoing research is completed. Performance data could be collected at the end of the maintenance period, or at some similar marker.
- For cement-stabilised material, the rate of change of in-situ modulus in the post-curing phase is dependent on several variables: elapsed ESA, reference modulus, and tensile strain and/or stress. Tensile strain itself will also increase in this phase, because the modulus decreases and this will likely increase the rate of deterioration. This behaviour is considered to apply to lightly bound materials,

while heavily bound materials may retain approximately constant stiffness, followed by a rapid decrease once cracking (usually block cracking) is initiated.

- For all bound materials, after sufficient trafficking the modulus is likely to degrade to a base value (E_b) of about 350–500MPa (isotropic). The rate of change of modulus is likely to relate to (E_c–E_b). Once the base value is reached, there will be a change to 'unbound second phase' life, during which performance will be largely dictated by the degree to which waterproofing is maintained, and the overall depth of the pavement above the subgrade.
- For foamed bitumen, it appears that a reference modulus also needs to be nominated, but this may be dependent on additional factors. At the time of this study, there were conflicting reports from researchers. Although it appears that foam-stabilised mixes are less sensitive to temperature variation than hot-mix asphalts, we believe they are still sensitive, and their moduli will change with the change of temperature. The modulus will also tend to increase slightly as the bitumen experiences environmental exposure (ageing). For these reasons, we proposed for this study that the reference modulus should be at one year, with no significant trafficking, and at 20°C. Then, when calculating the strains in the various layers, the modulus at the relevant WMAPT for the site could be used.
- The number of ESA to failure (cracking) for a bound layer is a 4-dimensional equation, rather than the 2-dimensional equation for unbound granular materials. Acknowledging this with reasonable modelling assumptions should significantly improve the reliability of pavement performance prediction of cement-treated materials.

Our conceptual model was set up and initially used to correct the moduli at all sites to E_c. This modulus was then used to find the horizontal tensile strain at the bottom of the cement-treated layer when subjected to a 1ESA load.

The conceptual equation is:

$$\mu\varepsilon = \frac{\frac{113{,}000}{{E_C}^{0.6}} + A}{\left(\frac{N_f}{RF}\right)^{\frac{1}{PF}}}$$

(Equation 5.3)

Where:

- A is a coefficient to maintain compatibility with Austroads (currently set at 0)
- N_f is the number of ESA inducing fatigue (since the time when curing was essentially complete) – this includes the SAR12/ESA factor of 3.6
- RF is the Project Reliability Factor (for 90% this is 2, for 95% this is 1)
- PF is the Power Law Factor, 9 – 1.5 * (Log10(Nf) – 4).

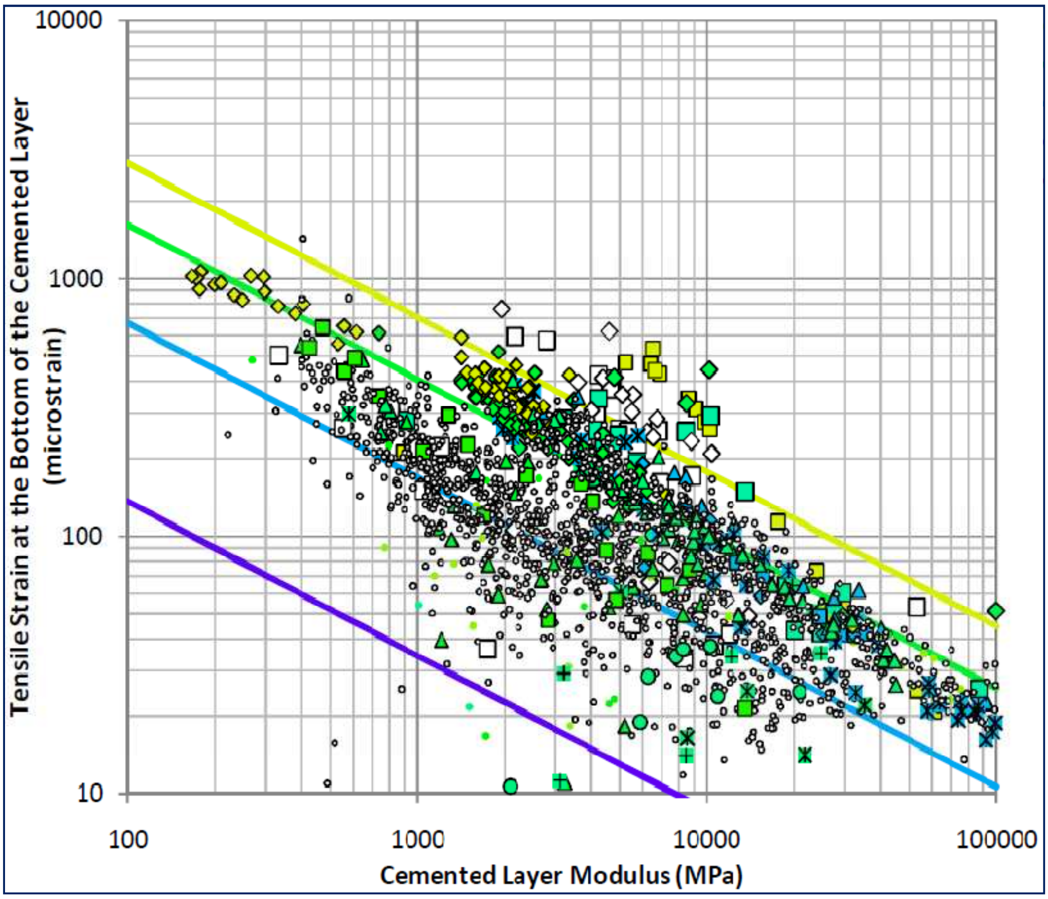
The best-fit PF is not constant in this model. It ranges from about 12 (ie a similar value to the current Austroads model) in pavements that have a low lifetime ESA in terms of resistance to cracking, to a much lower value (approaching 4) for pavements that are capable of high ESA when using this equation to fit the performance data. This finding appears to have some rationale, as it would be expected that a pavement that has high horizontal tensile strains (close to the ultimate tensile strength) will be much more sensitive to a doubling of traffic than one that has strains that are far from their ultimate values. (The PF is, after all, essentially a measure of a pavement's sensitivity to increased loading.) The graphical expression of this power law variability is that the spacing of the iso-ESA curves increases as the pavement lifetime ESA increases, rather than the equal spacing of the current Austroads model.

Our conceptual equation has more coefficients that the Austroads equation (Austroads 2008), but is no more time consuming to invoke in practice and allows much greater flexibility to match observed data. The inverse equation needs to be calculated iteratively at present.

Figure 5.13 shows the result of using reasonable coefficients in the conceptual equation to fit (in what was intended to be a conservative manner) the performance data in our existing inventory. The curve fit takes into account the observed failures at known sites (the large data markers in figure 5.13). Failure in this case usually means terminal distress in the form of cracking, followed by shallow shear and/or rutting in the active wheel paths as water initially enters the upper pavement, and then the need for maintenance intervention. The mathematical approach used with the conceptual pavement model is described in appendix C.

Ongoing refinement of the coefficients in the equation (or even restructuring the equation) will be helpful, based on ongoing collection and validation of performance data in documented case histories.

Figure 5.13 Conceptual performance curves for cement-stabilised pavements



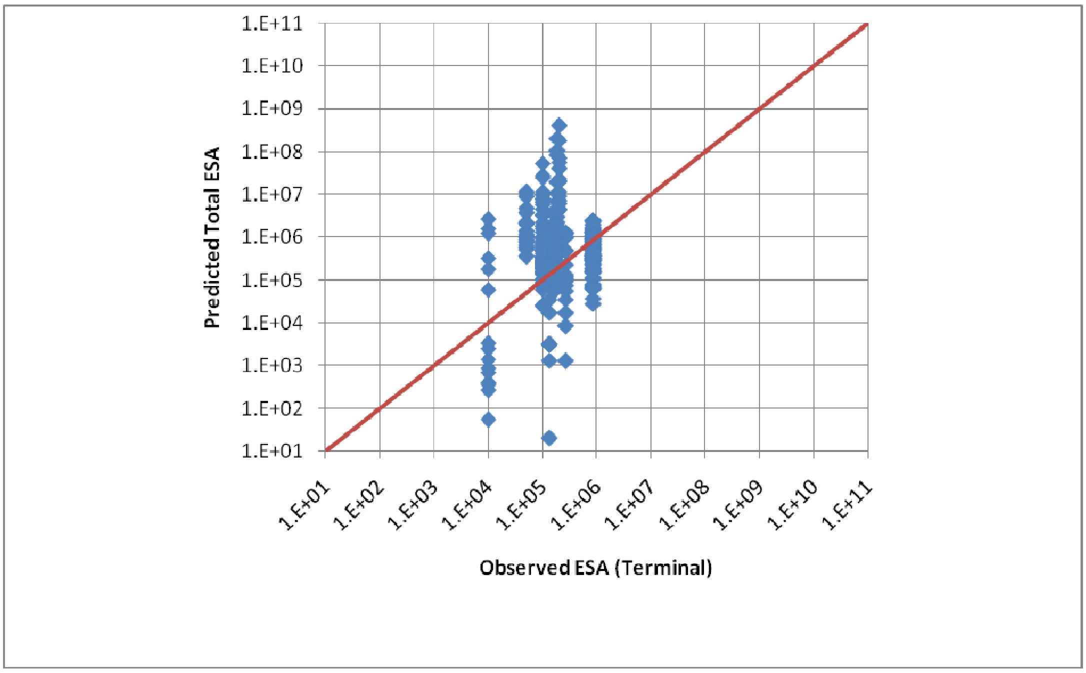
In the curve above, the conceptual performance curves range from 100,000 ESA (10^5ESA) (yellow line) to 10,000,000ESA (10^7ESA) (dark-blue line). The light-blue line represents the conceptual performance curve for 1,000,000ESA (1MESA or 10^6ESA). The curve for 100MEA (green line) plots below the x axis in the curve above.

It is important to note that when using the conceptual pavement model, the results are very dependent on having a reasonably correct age of the pavement at the date of deflection testing (ie to the nearest month if less than 1 year old, but thereafter to the nearest year). A reasonably correct estimate of elapsed ESA is also required.

The resulting observed life vs predicted life from the model shown in the above figure is shown in figure 5.14. The correlation is far from ideal, and in keeping with many other pavement life predictions, has to be plotted logarithmically to show any trend. However, the fit is an improvement over the one achieved by using the current Austroads fatigue equation (refer back to figure 5.10).

Therefore there is reasonable expectation that with more development, a practical model for performance prediction will result. Meanwhile the reported conceptual model should be sufficiently sound to allow reasonably reliable performance-ranking of stabilisation projects, relative to other projects with similar binder content and construction practices.

Figure 5.14 Spread of predicted pavement life vs observed pavement life from phase 4 modelling



5.6 Phase 5: Stress and strain relationships in cement-stabilised basecourse layers

Ongoing research by Dr Greg Arnold[9] suggests that for a cemented layer to withstand premature cracking, the tensile stress at the base of the layer should be less than the tensile strength for that material by a proportion. His '40% rule' suggests that cracking in a cemented-stabilised layer is avoided for traffic loading up to 1MESA if the stress in the cemented layer is <40% of the flexural beam tensile strength. He suggests that some researchers (including himself) found the tensile strength can be approximated as twice the ITS tensile strength. Hence we should aim to keep tensile stresses in the layer <ITS strength.

9 Pers comm 2011

Before investigating if this could be validated or otherwise by our performance data overall, we investigated how the performance of the Awaho Culvert site fared in this respect.

The upper pavement layer on the Awaho culvert project site (refer to section 4.7.4) was recycled in 1997/98. Since then it has carried around 1MESA.

Using the knowledge we had derived from this site (and other sites in the research project), we then investigated whether we could now use our research findings, and in particular, figure 5.13, to predict this level of performance (ie as a check that the model was functioning correctly, as this was a subset of the data from which the model was derived).

The best representation of the original pavement structure at the Awaho site was as follows:

- historic seal depth 50mm
- underlying pavement depth 350mm (giving a total pavement depth of 400mm)
- subgrade CBR 5%
- a design traffic loading of 1MESA (10^6ESA).

The design concept at the time was documented as follows:

- Overlay the existing pavement by 100mm (giving a total depth of 500mm), then stabilise the upper aggregate layers with cement and model with a Layer 1 design modulus (isotropic) of 2000MPa, which was used to represent the pavement layer over the 10yr+ life.

CIRCLY[10] can be used to assess the stress and strain conditions in this pavement under this scenario. In figure 5.15, the CIRCLY output comes from a pavement depth of 500mm over a subgrade CBR of 5%, and delivers a CDF of 107 in the cemented layer using the Austroads fatigue equation (Austroads 2008).

Figure 5.15 Pavement performance from CIRCLY analysis of Awaho Culvert project site

```
Details of Layered System:

   ID: Awaho Title: Awaho Culvert SH 2 Recycling Review

   Layer  Lower    Material      Isotropy   Modulus    P.Ratio
    No.   i/face   ID                       (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Cement2000    Iso.       2.00E+03   0.20
    2     rough    Gran_250      Aniso.     2.50E+02   0.35      1.90E+02   1.25E+02   0.35
    3     rough    Sub_CBR5      Aniso.     5.00E+01   0.45      3.45E+01   2.50E+01   0.45

   Performance Relationships:
   Layer  Location Performance   Component  Perform.   Perform.  Traffic
    No.            ID                       Constant   Exponent  Multiplier
    1     bottom   Cement2000    ETH         0.000442   12.000     3.600
    3     top      Sub_2004      EZZ         0.009300    7.000     1.200

   Reliability Factors: Not Used.

   Details of Layers to be sublayered:
   Layer no.  2:  Austroads (2004) sublayering

Results:

   Layer  Thickness  Material     Load         Critical       CDF
    No.              ID           ID           Strain
    1       250.00   Cement2000   ESA75-Full   -1.85E-04      1.07E+02
    2       250.00   Gran_250                   n/a           n/a
    3         0.00   Sub_CBR5     ESA75-Full    4.19E-04      4.49E-04
```

10 CIRCLY Ver 5.0r: MINCAD systems

The CDF of 107 (see the above figure) suggests that the upper 250mm cement-stabilised basecourse pavement layer can be expected to fail by cracking after only 10,000ESA following construction. However, we know that this pavement has performed adequately for more than 10 years. The very low CDF for the subgrade (0.0005) is indicative of a well-protected subgrade, which was a prerequisite for the selection of the recycling option employed on this project.

The output from the CIRCLY analysis of this project also predicts maximum tensile strain at the base of the cement-stabilised layer of around 180με, as shown in figure 5.16.

Investigating the stress levels in figure 5.16) allowed us to compare these with the likely tensile strength of this cemented-stabilised material. Tensile stresses at the base of the cement-stabilised layer were around 400kPa. We do not have ITS test results for this project. However, the data presented in figure 4.4 suggests that the average ITS strength for the samples with 3% cement binder is around 400MPa. The stress levels appear to be consistent with an expected pavement life according to Arnold (pers comm), as discussed at the beginning of this section.

Figure 5.16 Stabilised-layer performances from CIRCLY analysis of Awaho Culvert

Awaho Culvert SH 2.clo

ESA750-Full

	POINT	COORDINATES				DISPLACEMENTS		
	NO.	X	Y	Z	L	UX	UY	UZ
0	1	-0.1650E+03	0.0000E+00	0.2500E+03	1	0.2437E-01	0.2130E-08	-0.6179E+00
0	2	0.0000E+00	0.0000E+00	0.2500E+03	1	0.7975E-02	0.2001E-08	-0.6447E+00
0	3	0.1650E+03	0.0000E+00	0.2500E+03	1	-0.8074E-02	0.8777E-09	-0.6473E+00
0	4	-0.1650E+03	0.0000E+00	0.5000E+03	7	0.4999E-01	0.4370E-08	-0.5469E+00
0	5	0.0000E+00	0.0000E+00	0.5000E+03	7	0.2482E-01	0.3769E-08	-0.5698E+00
0	6	0.1650E+03	0.0000E+00	0.5000E+03	7	0.3391E-03	0.2583E-08	-0.5755E+00

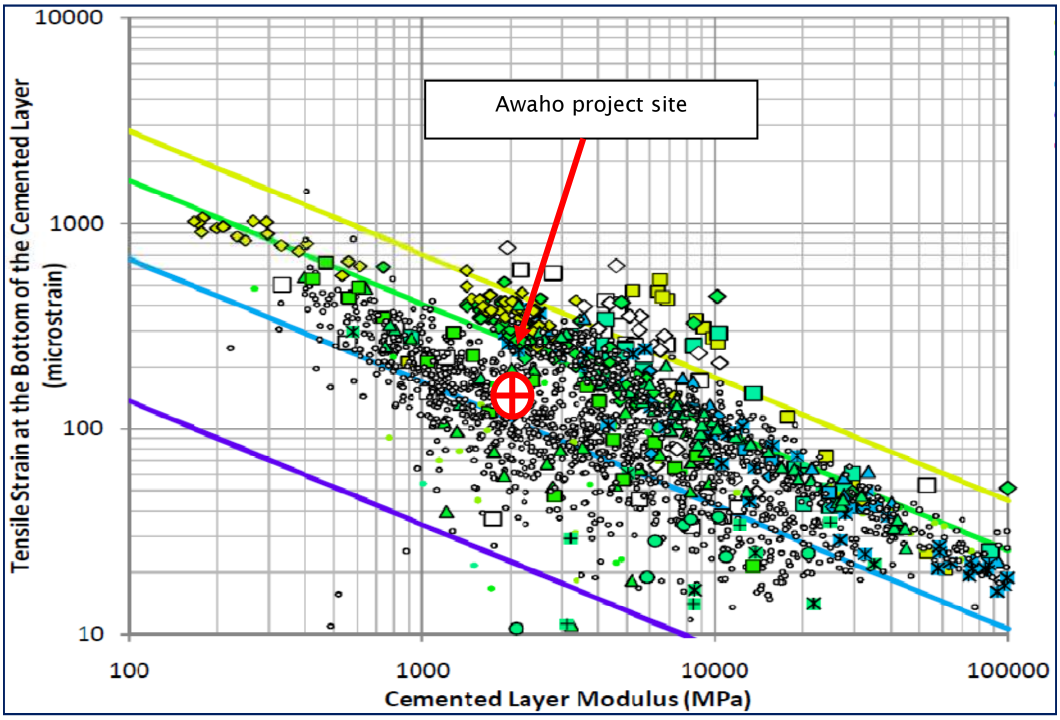
	POINT	COORDINATES				NORMAL STRAINS		
	NO.	X	Y	Z	L	XX	YY	ZZ
0	1	-0.1650E+03	0.0000E+00	0.2500E+03	1	-0.1080E-03	-0.1737E-03	0.8969E-04
0	2	0.0000E+00	0.0000E+00	0.2500E+03	1	-0.8867E-04	-0.1853E-03	0.8708E-04
0	3	0.1650E+03	0.0000E+00	0.2500E+03	1	-0.1032E-03	-0.1767E-03	0.8945E-04
0	4	-0.1650E+03	0.0000E+00	0.5000E+03	7	-0.1403E-03	-0.2206E-03	0.3877E-03
0	5	0.0000E+00	0.0000E+00	0.5000E+03	7	-0.1574E-03	-0.2359E-03	0.4185E-03
0	6	0.1650E+03	0.0000E+00	0.5000E+03	7	-0.1319E-03	-0.2284E-03	0.3940E-03

	POINT	COORDINATES				NORMAL STRESSES		
	NO.	X	Y	Z	L	XX	YY	ZZ
0	1	-0.1650E+03	0.0000E+00	0.2500E+03	1	-0.2865E+00	-0.3961E+00	0.4285E-01
0	2	0.0000E+00	0.0000E+00	0.2500E+03	1	-0.2516E+00	-0.4127E+00	0.4130E-01
0	3	0.1650E+03	0.0000E+00	0.2500E+03	1	-0.2779E+00	-0.4004E+00	0.4322E-01
0	4	-0.1650E+03	0.0000E+00	0.5000E+03	7	0.2596E-03	-0.1124E-02	0.1900E-01
0	5	0.0000E+00	0.0000E+00	0.5000E+03	7	0.7815E-04	-0.1276E-02	0.2039E-01
0	6	0.1650E+03	0.0000E+00	0.5000E+03	7	0.6264E-03	-0.1037E-02	0.1951E-01

It is important to remember that the cement-stabilisation work at the Awaho Culvert project was used to correct surface-layer instability problems, not deep-seated pavement defects, as suggested by the low CDF for the subgrade rutting in figure 5.16. The existing pavement was failing in shallow shear and short seal cycles/SCRIM, and maintenance intervention was needed. Recycling of the near-surface basecourse/seal using cement stabilisation was a cost-effective outcome compared with premium granular overlay.

Using a layer modulus of 2000MPa and an expected tensile strain level of 180με in figure 5.17 (following) delivers an expected pavement design life of $0.8*10^6$ESA. From a pavement designer's perspective, this result suggests that if effective construction delivers the expected cemented-layer modulus, and then effective maintenance operations allow any micro-cracking to 'self heal', then for stabilised-layer modulus of 2000MPa, figure 5.17 (ie the figure based on our conceptual pavement model) would deliver a theoretical pavement design life of around 1MESA, which is as we have observed.

Figure 5.17 Conceptual pavement performance modelling of Awaho Culvert project



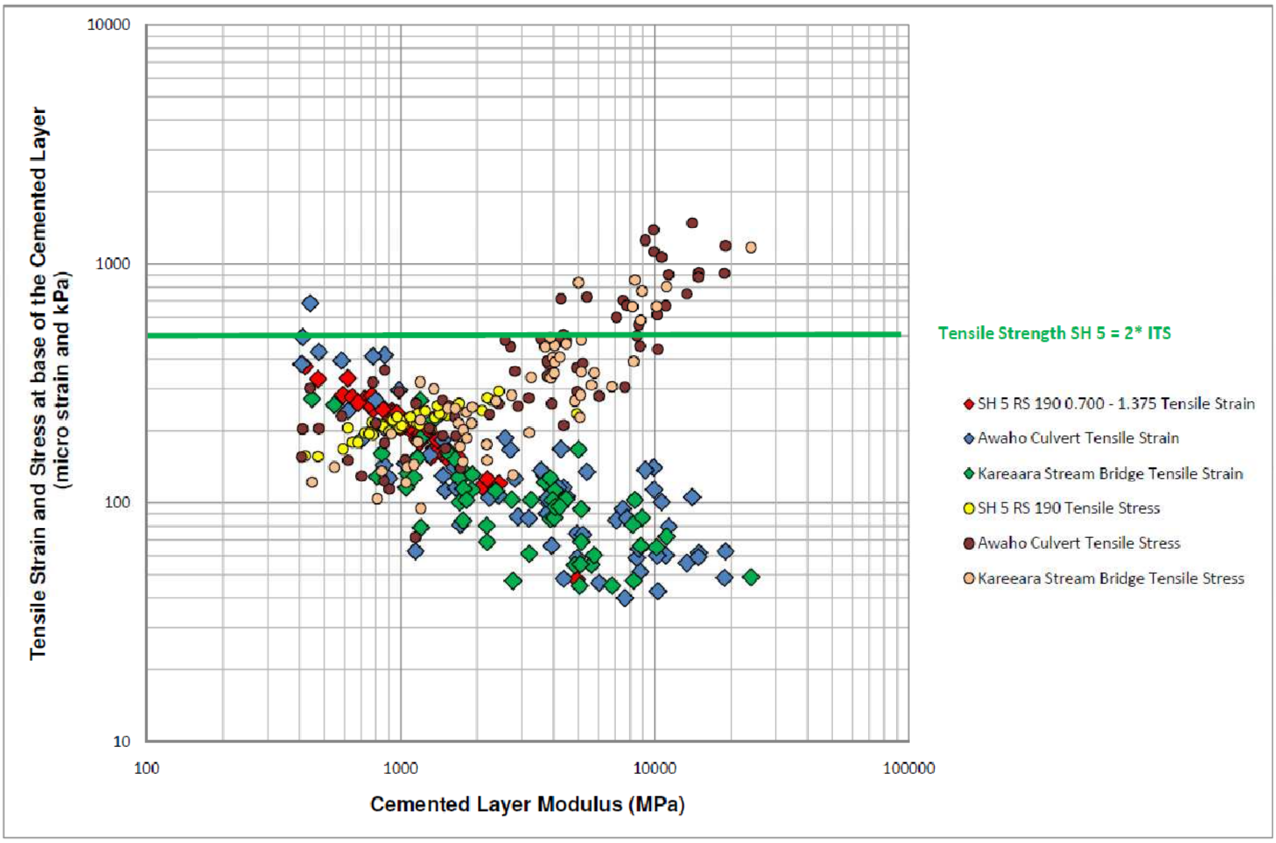
We then moved on to investigate in more detail the stress conditions likely within the cement-stabilised materials reported in our data inventory.

First, we investigated whether we could extract the pavement-layer stress conditions from the FWD analysis completed using ELMOD (refer to section 3.1). We concluded that there may be a technical hitch here. The Odemark/Bousinesq equation for 3-dimensional (3D) tensile stress is not the same as the linear case; ie it depends only on the vertical stress at the surface, depth and Poisson's ratio, not on modulus E (Ullidtz 1987). Since most of our inventory pavements have a similar stabilised-layer thickness (around 200–250mm compacted-layer thickness) with the same Poisson's ratio and ESA load at the surface, the tensile stress at the bottom of the layer should be much the same. However, tensile strain differs, because the 3D case (which again is not as simple as linear) is a function of modulus, Poisson's ratio, vertical stress and vertical strain.

In response to this 'challenge', we then used the back-calculated modulus and horizontal tensile strain from the deflection bowl to generate a 'pseudo-stress' at the base of the stabilised layer given by a nominal modulus times strain parameter. This was not at all equivalent to the tensile stress beneath the 1ESA load, but it did give us a parameter (with dimensions of stress) to explore its possible relevance to the observed pavement performance.

In figure 5.18 we examine the likely stress/strain conditions in the cemented-stabilised basecourse layer near the beginning of the stabilised pavement life, using the results of our analysis for site 7 on SH 5 between Napier and Taupo, and compare these with the results from the Awaho and Kareeara project sites (refer to section 4.3).

Figure 5.18 Stress and strain conditions for selected projects



Based on this simplified model, we observe the following:

- For the most recent construction work (SH 5 RS 190 0.7 – 1.375, constructed in early 2010), both the stress and strain results were located in what appeared to be a stable performance zone, based on our research findings. The pavement-layer stresses were approximately 50% of the assumed tensile stress based on the measured ITS of 253kPa (tensile strength of 500kPa, shown as the green line) for this site.
- For the Awaho and Kareeara sites, the stress and strain results shown in the above figure represented the pavement condition soon after construction. Up to the layer of modulus of 3000MPa, the behaviour was similar to the SH 5 site. For layer modulus >3000MPa, the stress results seemed to deviate from our expectation that the stress conditions should be similar for pavement layers of similar depth.

In table 5.2 we present the percentile values for layer modulus, tensile strain, and subsequent tensile stress for the SH 5 site shown in the above figure. The 90th percentile tensile stress is 53% of the assumed tensile strength, based on twice the measured ITS from the recently cored sample.

Some background information about this site:

- *AWPT treatment on SH 5 for SCRIM*, 100mm overlay, followed by top 250mm stabiliser 3% cement on overall pavement of 400mm
- *Current condition* good, with no obvious defects requiring intervention.

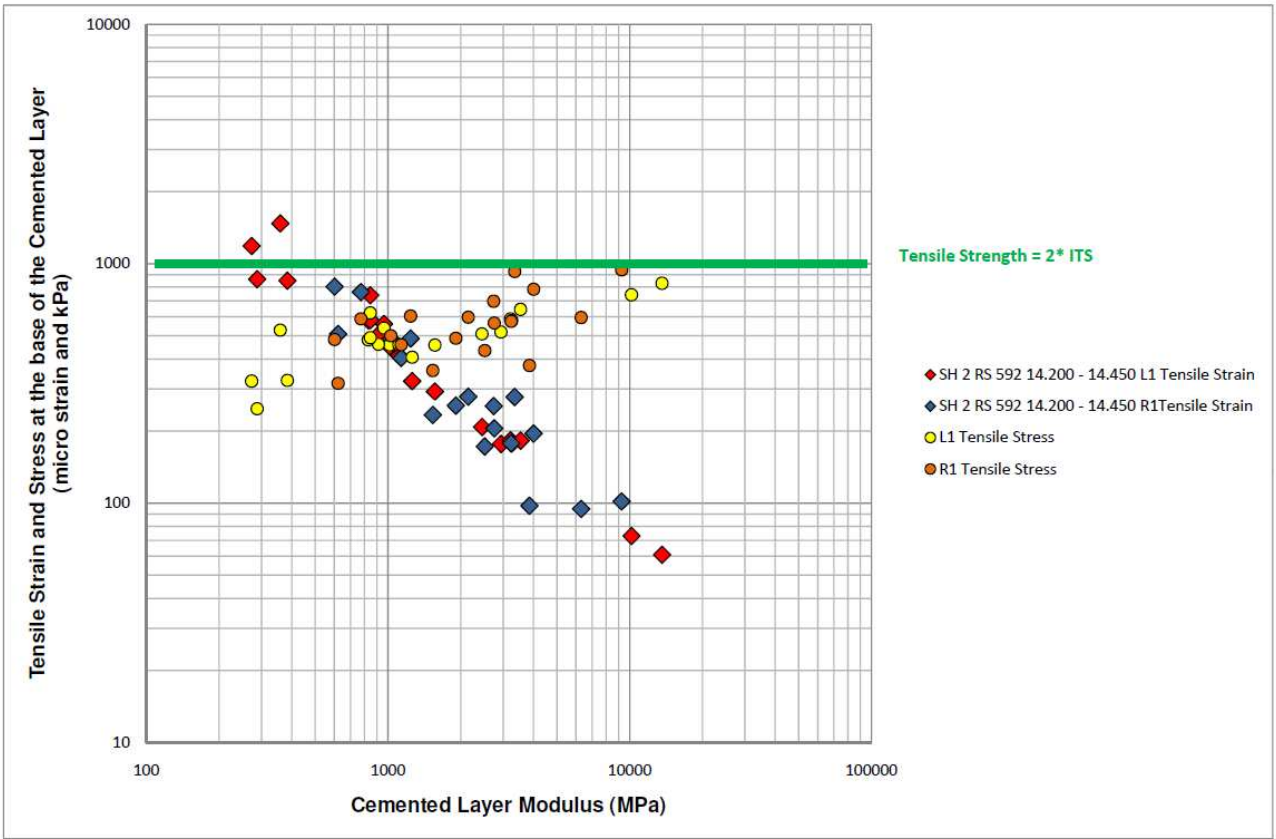
The NZTA/Opus in Napier evaluated this site as being suitable for 'recycling with top-up' in 2009. The data presented above would appear to justify this treatment option.

Table 5.2 Statistical analysis of results for SH 5 RS 190 in figure 5.18

Parameter	10%ile	50%ile	90%ile
Modulus	656	1119	2256
Strain	122	204	280
Stress	183	223	266

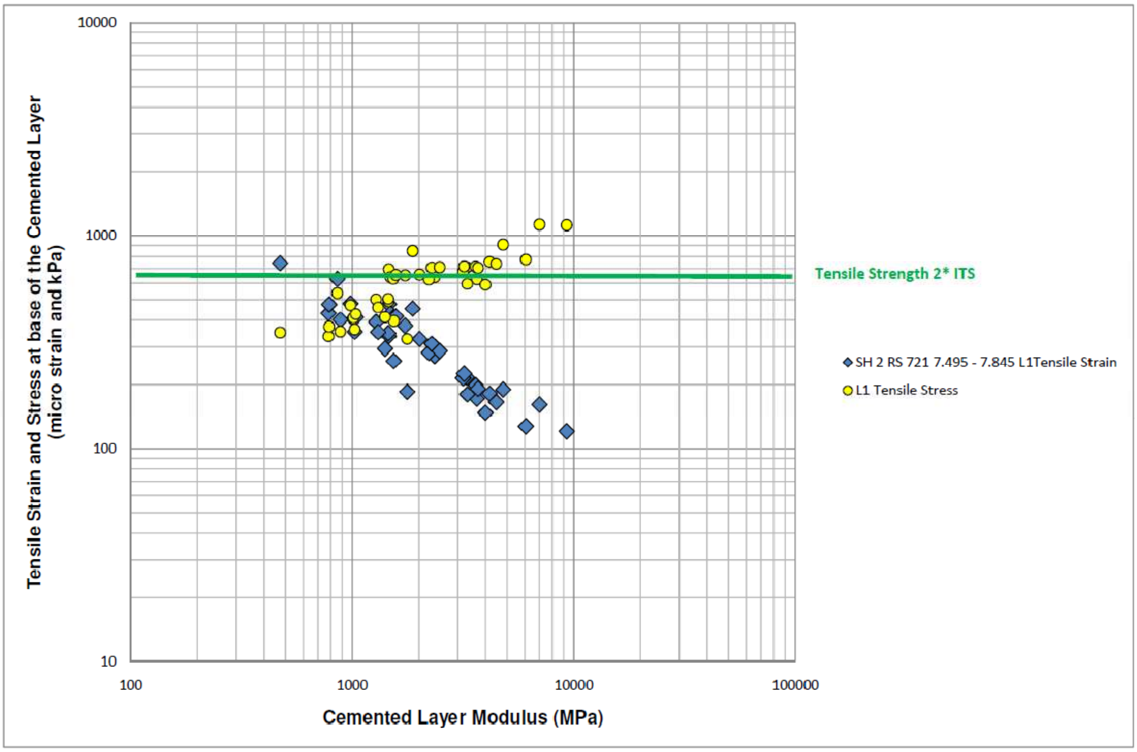
We then took other data from the inventory and investigated the likely stress/strain behaviour in the stabilised layers at other project sites, as shown below in figures 5.19–5.20.

Figure 5.19 Layer stress and strain conditions for site 9 (SH 2 RS 592 14.2 – 14.45)



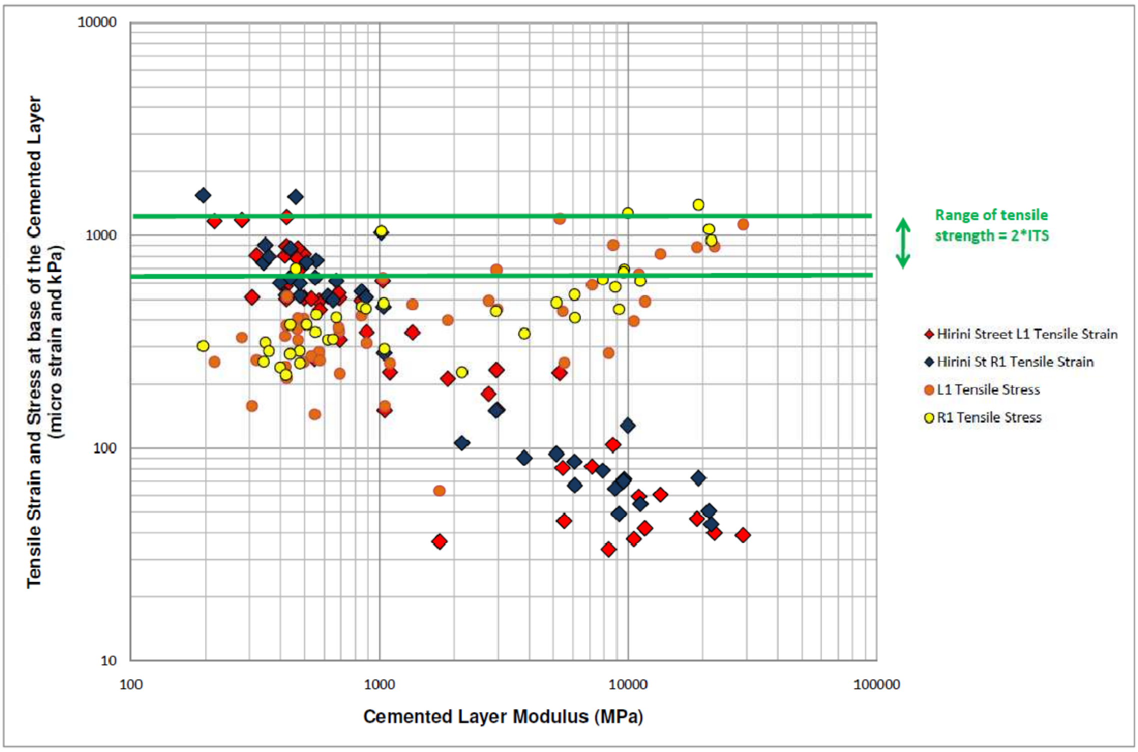
At the time of this research, site 9 required urgent maintenance intervention for wheeltrack cracking and shallow shear problems. The pavement-layer stress conditions here appear to lie between 50% and 100% of the assumed layer tensile strength.

Figure 5.20 Layer stress and strain conditions for site 15 (SH 2 RS 721 7.495 – 7.845)



At the time of this research, site 15 was in a relatively stable condition after nearly 14 years. The pavement layer stress conditions here appear to lie between 75% and over 100% of the assumed layer tensile strength. The latter was based on 2*ITS (ITS of 318kPa). These results seem to suggest that a change in pavement condition is imminent.

Figure 5.21 Current stress and strain conditions for site 16 (Hirini St)



Site 16 had failed over approximately 30% of the left-hand lane (L1) soon after construction, and this required urgent maintenance intervention. The back-calculated modulus and strain outcomes will have been affected by the distressed condition of the pavement in a number of locations.

In order to further explore the possible relationship between lightly bound pavement performance and tensile stress/strength, we took the measured performance of site 11 (SH 2 RS 608 15.730 – 17.130 Kareeara, see figure 5.21) and plotted the back-calculated cemented-layer modulus and tensile strain, and then calculated tensile stress behaviour post-construction and in the current condition some 14 years later. We then compared the tensile stresses in the layer with the tensile strength of the cement-stabilised material. To put this site into perspective, the photograph in figure 5.22 shows a recent high-speed data photograph from this hill-country site just north of Napier.

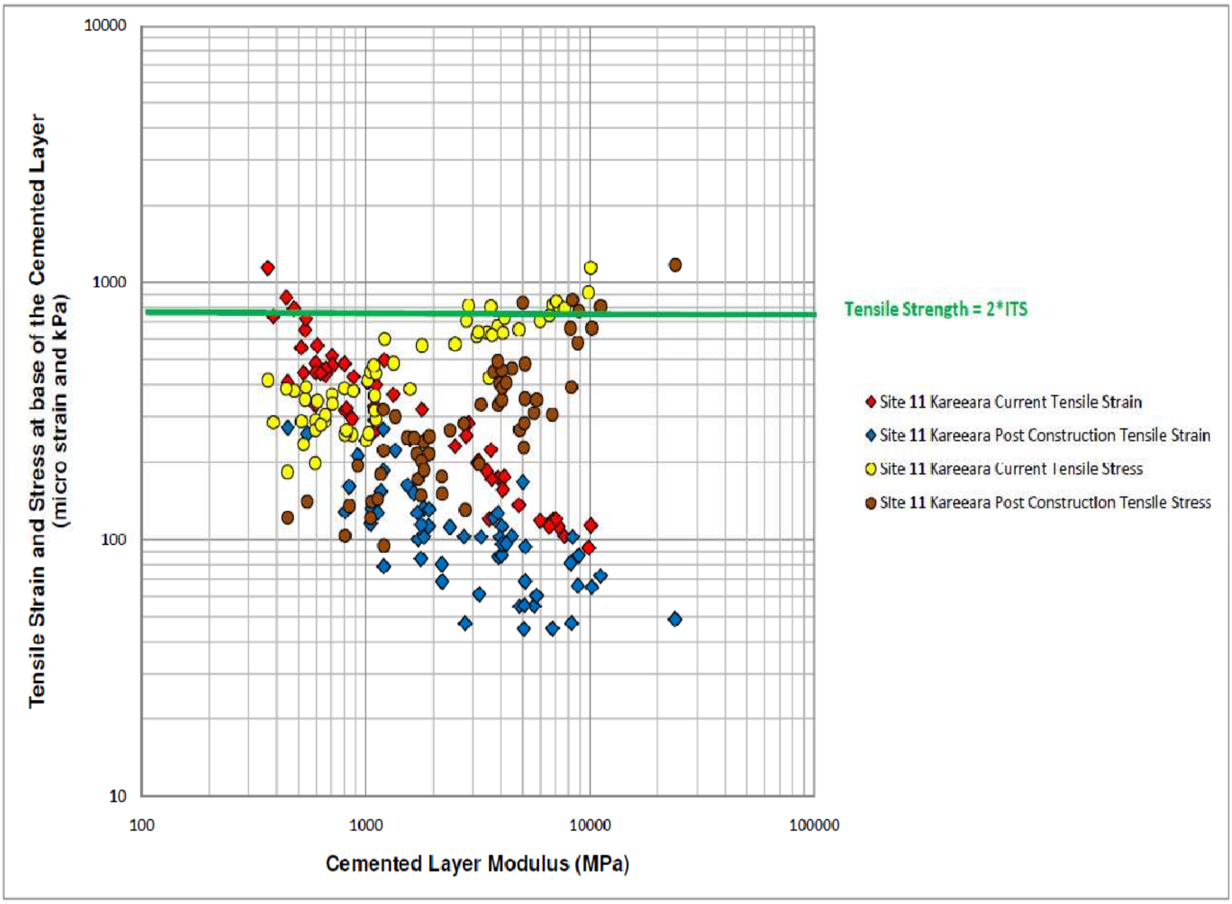
Figure 5.22 Site 11: SH 2 RS 608 15.703 – 17.130 – constructed in 1997

The calculated post-construction tensile stresses in the cemented basecourse were generally about or less than 50% of the tensile strength (based on recent cored ITS testing) for the range of modulus up to 5000MPa, as shown in figure 5.22.

It could probably be argued that the post-construction tensile strength would have been higher following curing, making this comparison even more favourable.

Since construction, the stresses in the cemented pavement layer appeared to be generally trending higher over time, mirroring an increase in layer strain as well. This change in behaviour was expected as the cemented layer transitioned from lightly bound to unbound, as reported previously in section 5.2.

Figure 5.23 Layer stress and strain conditions for site 11 (Kareeara)



5.7 Summary of research findings from section 5

The research findings presented in this section describe a journey of sorts. Our research team used the pavement performance data in our data inventory to explore the use of a conceptual pavement model that would allow designers to better predict the performance of pavements with lightly bound cement-stabilised basecourse layers. Performance data from pavements utilising such materials from around New Zealand clearly indicated that the expected pavement performance exceeded that predicted by the current Austroads fatigue equations.

Our conceptual model delivered interim performance curves linking layer modulus and tensile strain with design traffic (see figure 5.13).

This research and research by others (Arnold pers comm) suggest that in pavements using cement-stabilised lightly bound upper-pavement layers, whilst the overall pavement depth can be investigated using current theory (Austroads 2008 and Transit NZ 2007), a check should also be made of tensile stresses within the stabilised layer, using elastic theory (CIRCLY). It appears that if the tensile stresses within the stabilised layer are kept below 50% of the tensile strength of the material, then fatigue cracking will not govern behaviour. As a first approximation, the tensile tresses in the stabilised layer should be <ITS.

6 Characterising and modelling modified, lightly bound and bound pavement layer behaviour

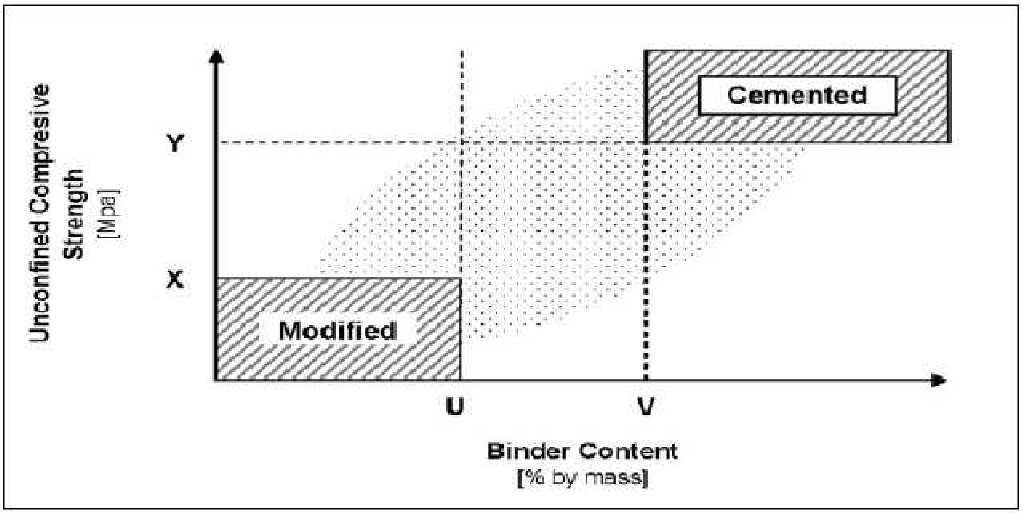
Our research demonstrated that near-surface basecourse aggregate materials (including existing seal materials and any overlay aggregate, when appropriate), when stabilised with cement, are more likely to become either lightly bound or bound, depending on the quantity of additive used. Previously, stabilisation with small quantities of cement (<3% by dry mass) was assumed by most pavement designers to deliver a modified material that could be modelled as unbound.

The implications of this finding are useful in the New Zealand context for the following reasons:

- Stabilised basecourse materials contribute to cost-effective treatment options in rehabilitation projects (eg recycling) and in maintenance works, and help to mitigate against premature wheeltrack rutting in new pavement solutions.
- Our improved understanding of how these pavement solutions perform over time (eg lightly bound materials can be modelled with layer moduli between 700MPa and 5000MPa and will successfully deliver >1MESA design traffic, provided layer strain and stress levels are managed) provides pavement designers with a credible design alternative
- The investigation and design of pavement solutions utilising lightly bound materials could, in future, be described with more confidence in pavement design guidelines used in New Zealand.

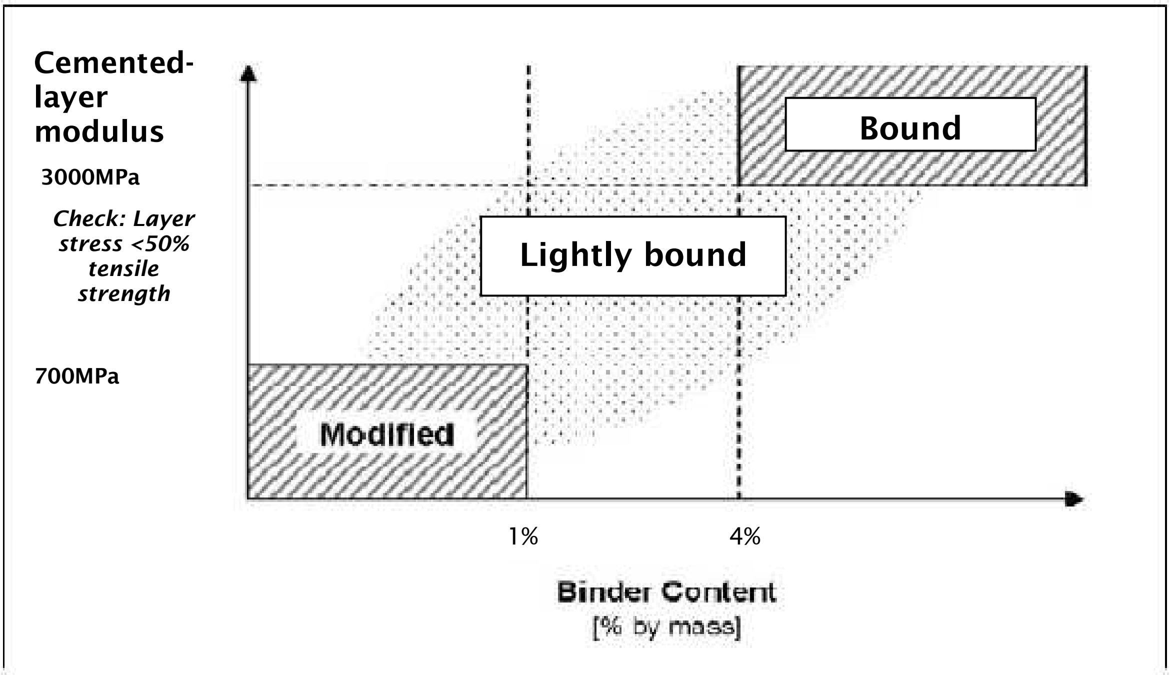
In section 1.3 of this report we included the graph reproduced as figure 6.1 below. This describes the current industry understanding of the relationship between modified and cemented materials, based on binder content and strength. It highlights the significant area of uncertainty that lies between the two.

Figure 6.1 Current characterisation of modified and bound (cemented) materials



The objective of this research project was to provide guidance to the wider transport industry, based on actual performance data from stabilised pavement solutions in New Zealand, on the characterisation and effective use of stabilised granular materials in pavement projects. We used the outcomes of this research to present more information on the concepts of lightly bound materials, as shown in figure 6.2.

Figure 6.2 Characterisation of cement-stabilised aggregate pavement layers



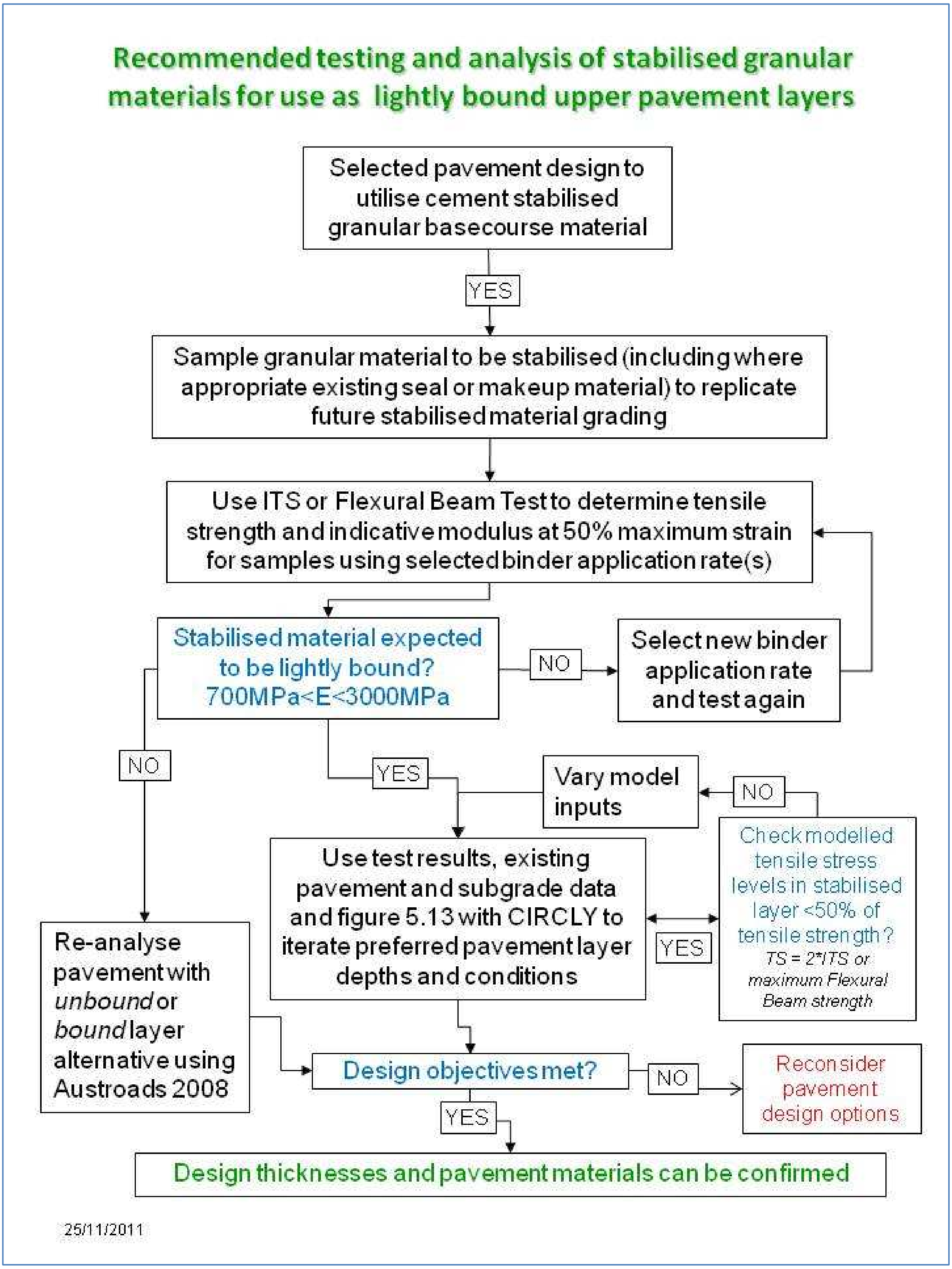
The concepts presented in the above figure can be summarised as follows:

- When using cement additive contents of between 1% and 3% by dry mass in stabilised basecourse materials (with or without seal inclusion) the resulting cement-stabilised pavement layer should be modelled as a lightly bound material.
- The design of pavements incorporating lightly bound cement-stabilised basecourse layers can utilise a conceptual pavement model (developed from this research) that incorporates the resilient modulus of the cement-stabilised material after curing, design traffic loading and maximum allowable tensile strain.
- When designing the pavement, the overall pavement depth should be determined using current published guidelines (Austroads 2008 and Transit NZ 2007). Then a check should also be made of tensile stress within the lightly bound layer. Research indicates that if the tensile stresses within the stabilised layer are kept below 50% of the tensile strength of the material, or at or less than the minimum ITS from samples prepared using representative pavement materials, then fatigue cracking will not govern behaviour.
- The design of a pavement utilising a cement-stabilised lightly bound basecourse layer can be completed by modelling using CIRCLY and iterating pavement depth and layer strength to meet the constraints on stress and strain described above.

Truly modified materials should continue to be assessed as though they were unbound pavements. Truly bound materials should continue o be assessed using published guidelines (Austroads 2008 and Transit NZ 2007).

The concepts described above have been assembled into the conceptual flowchart shown in figure 6.3.

Figure 6.3 Flowchart to support testing and analysis of lightly bound pavement layers



7 Quantifying pavement life for foamed bitumen/cement-modified pavement layers

7.1 Introduction

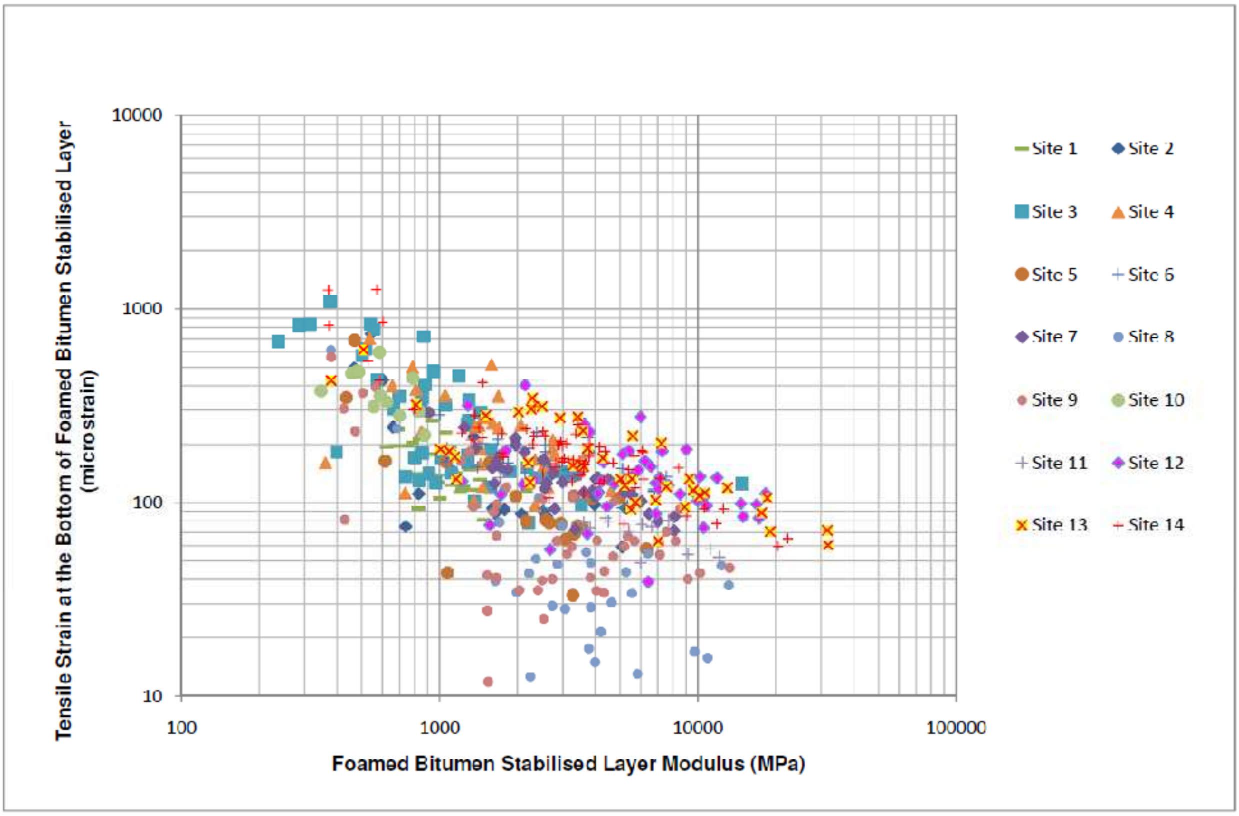
The development of performance criteria for FBS is significantly more challenging at this stage. The relatively recent introduction of this construction method into New Zealand means that there are a limited number of constructed sections. At the time of this research, there were no sites in our database with a service life of more than six years.

In section 4.3 we presented three pavement projects that utilised FBS. Two of these sites were on SH 1 north of Taupo. In these projects, the foamed bitumen/cement construction was completed in winter, under cold, wet conditions. The existing dacite granular basecourse layer was overlaid with <100mm of make-up metal (to contribute to granular make-up in the stabilised layer), stabilised and sealed. In one case, premature seal failure required remedial works. These FBS sites were continuing to perform well. There was no evidence of ongoing pavement deterioration. The third site was on SH 5 between Taupo and Napier. This again was a stabilised dacite aggregate overlay and was showing no sign of deterioration.

7.2 Performance review

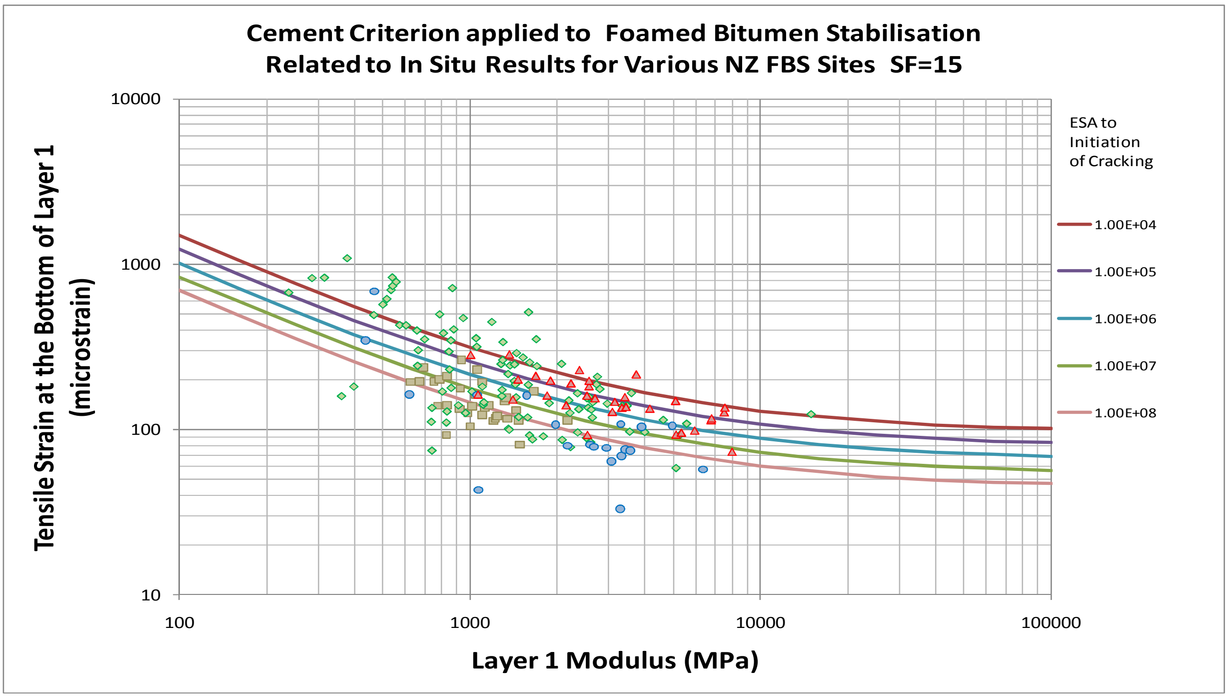
In figure 7.1 we have reproduced back-calculated performance data from the sites in our larger data inventory that utilised FBS. No failures were evident at these sites at the time of publishing this research.

Figure 7.1 FBS project data



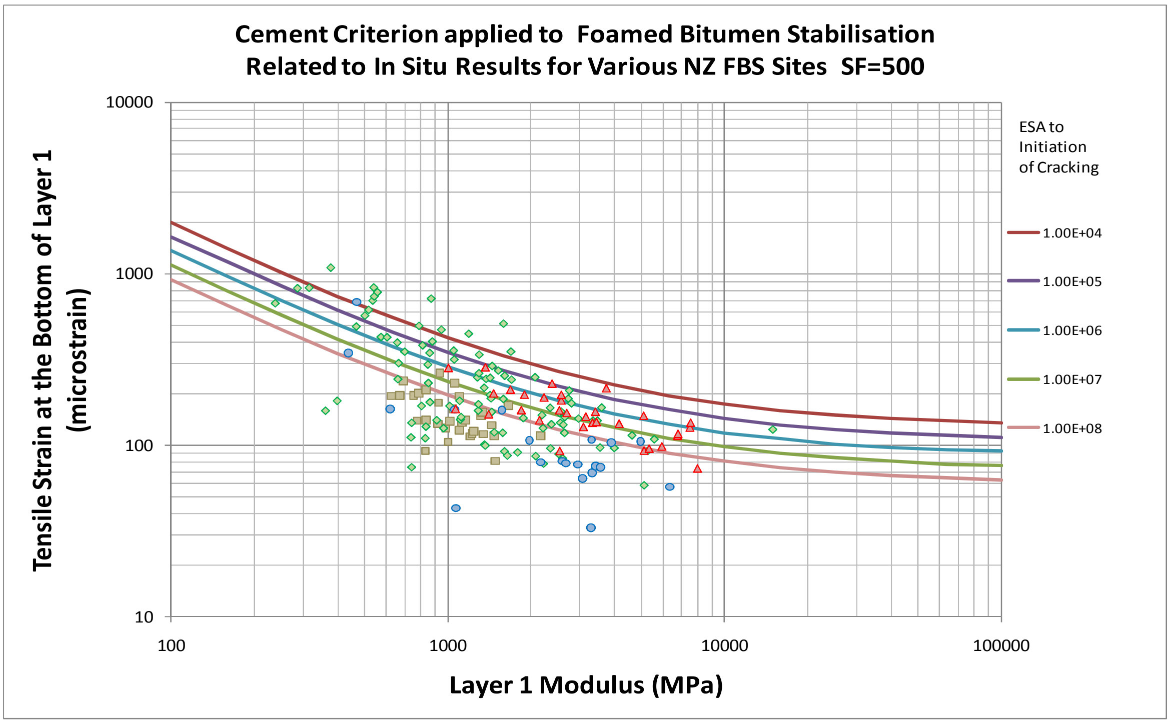
We initially postulated that as there was some cement in the mix, it could be a reasonable starting place to use the Austroads equation 6.4 (Austroads 2008) that was presented in section 5.3 of this report. We hoped to find a value of SF that made reasonable sense in relation to the plot of the layer 1 modulus and the tensile strain at the bottom of the layer, for all areas that were performing well (see figure 7.2).

Figure 7.2 Conceptual performance curves for FBS



Then for those same sites, we used a significantly higher SF to better position the ESA to initiation of cracking; ie SF=500, as shown in figure 7.3.

Figure 7.3 Conceptual performance curves for FBS

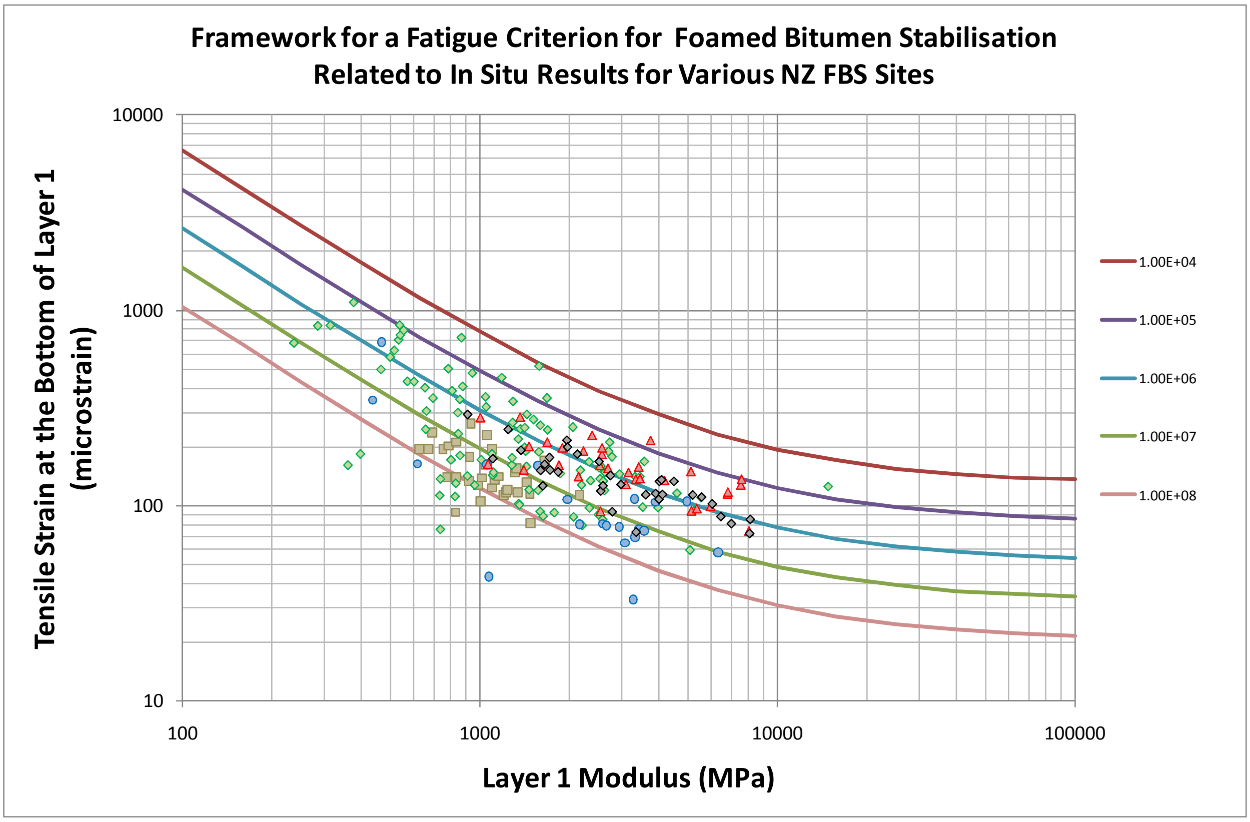


On this basis, it seems that the Austroads cement-bound criterion is presenting an ESA to initiation of cracking that is too conservative for FBS materials. To date, the adjusted SF of 500 provides a more appropriate positioning of test data next to field performance.

This is perhaps supported by current design philosophy for FBS, where the cement content is limited to no more than 1.5% (and typically lies between 1% and 1.5%), so that the primary failure mode is ductile rather than fatigue cracking. To date, reported and observed performance suggests that where cement content has been limited to no more than 1.5% by weight, fatigue cracking has not developed. However, further research and monitoring through design life loading is required to validate this assumption and approach.

Next, as there is no real reason to use the same constants as in the Austroads equation 6.4, the figure below provides a general fit to the data, spacing out the lines more, as the FBS material is likely to follow a power law much less than the 12-value adopted for Austroads use with cemented materials.

Figure 7.4 Conjectural fit for FBS life prediction, unconstrained by Austroads equation 6.4 constants



This increased spacing still suggests that a reasonable proportion of the test data is positioned above the ESA to initiation of cracking design loading, and further evaluation of field performance is required to 'calibrate' this model.

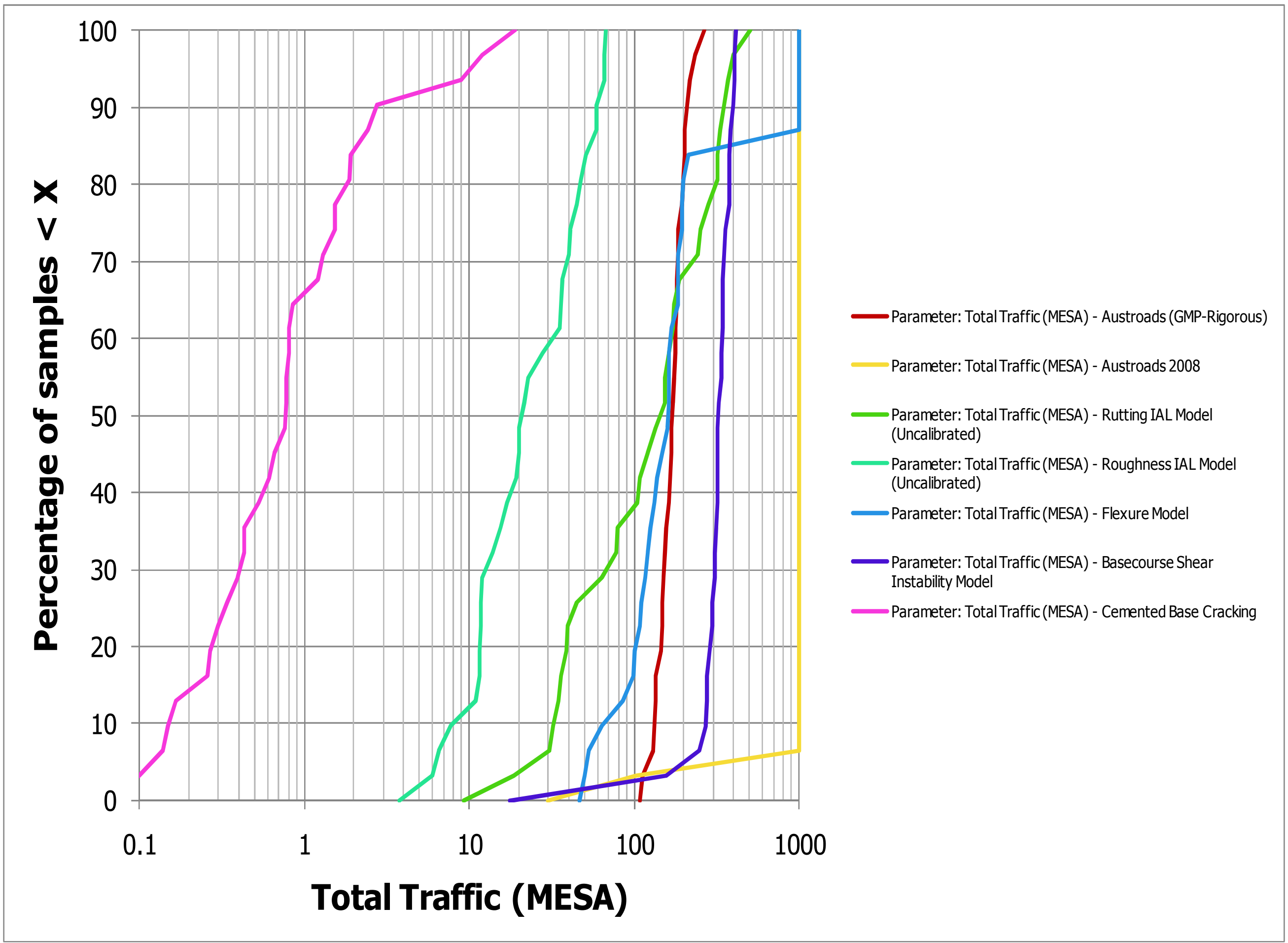
Until we get a lot more than this small handful of only six case histories *and* some points that demonstrate failure at known traffic loading, everything here is conjecture. However, this work does provide a framework to establish baseline fatigue criteria for ongoing use in the next few years as further performance data is generated.

It is useful to plot the data in this manner to see whether or not the data 'falls' on the high side of points known to be performing well. In figure 7.4 we can see that the red and black points (from two recent projects that are showing some initial sign of distress (unnamed here to avoid any contention at this speculative stage) come into this category. It will therefore be worthwhile to keep a close eye on these particular projects, as if they do not, in future, show failure by cracking prematurely, it may well prove that the limits can be pushed further out (ie a greater strain can be accommodated by the FBS layer than postulated here).

The site shown with black points in figure 7.4 is an interesting one – while there are a few irregularities in the pavement surface, it is not demonstrating failure per se. It appears there are a few localised low-shear-strength zones where previous unsuitable materials weren't dug out – and it also has a poor surface ride due to construction faults. However, there is definitely no cracking distress, and the great majority of the pavement is performing as expected. Again, it will be worthwhile to keep a close eye on this particular project, as it may well prove that the limits can be pushed further out (ie a greater strain can be accommodated by the FBS layer than postulated here).

In figure 7.5 we present the conceptual distress-life graph for the red points in figure 7.4, based on the FWD back-analysis.

Figure 7.5 Distress-life graphs for 'red points' in figure 7.4



This figure demonstrates that if the pavement were cement-stabilised only (rather than supplemented with foamed bitumen), cemented base cracking (the pink line above) would be the critical failure mode – where the 10th percentile capacity to failure is around 150,000 standard axles, with roughness the next critical failure mode, with a 10th percentile capacity to failure around 8MESA. Ongoing condition monitoring of the site against loading experienced since rehabilitation would eventually allow establishment of an FBS tensile-strain criterion in place of the cement-stabilised criterion that is being used as a nominal (very conservative) substitute in the model meanwhile.

We emphasise that all the discussion above is simply a *framework* to permit the predicted life of these pavements to be assessed by comparing performance (and distress modes, where manifested) against measured modulus and strains in the future.

There is little doubt that these figures are likely to need SFs applied to them, and some may be large, but the *relative* performance should be reasonably consistent. For example, points showing minimum cracking resistance on any one section should crack first, and points showing minimum rutting resistance should show deformation first. Ongoing site monitoring will permit the SF for the individual distress modes to be validated.

After this report is published, the objective must be to continue gathering data at selected sites for continued refinement of the various distress modes. There are differing schools of thought internationally with respect to the appropriate means of modelling FBS materials, and the critical failure mode is of particular interest. This is largely subject to the proportion of bitumen and active filler, and the overall pavement system. What has been established internationally is the importance of a robust anvil underneath the FBS layer, where it is preferred that the modular ratio of FBS to underlying sub-base layer modulus is no more than 5[11].

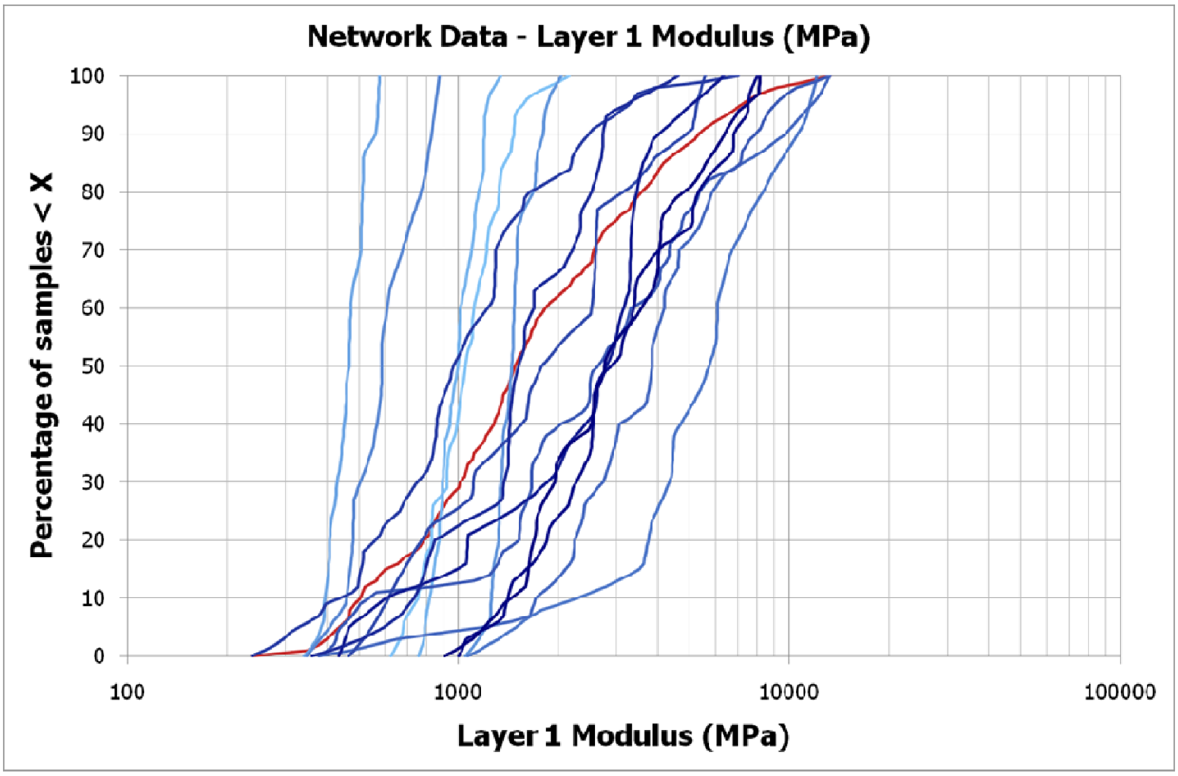
At the time of writing, we concluded that so far in New Zealand there was only a limited data pool for FBS project sites, a maximum of five years' service life, and a lack of confirmed failure modes, which all prevented us from confirming a conceptual performance model. Therefore it was not yet possible to define limit strains or characteristic performance criteria.

However, the evaluation undertaken through this research project, and the wider industry research/construction feedback we have received, suggest that the recommended NZTA design parameters for FBS (Transit NZ, 2007) are reasonable, provided the substrate top sub-layer stiffness is sufficient to limit the modular ratio to no more than 5, as outlined above.

The current New Zealand design parameters allow the FBS layer to be modelled as an isotropic layer with modulus of 800MPa. We took the inventory data and investigated the back-calculated FBS layer moduli as shown in figure 7.6. We note that the moduli for the FBS layers are lower than we thought they would be (we thought that the 10th percentile modulus would be around 800MPa as is assumed for design, but it is closer to 500MPa, 800MPa being closer to the 20th percentile). This finding reinforces our recommendation above, that after this report is published, an objective for the NZTA should be to continue gathering data at selected FBS sites to enable continued refinement of the various distress modes and design recommendations.

11 Allen Browne, Hiway Stabilizers – pers comm, July 2011

Figure 7.6 Back-calculated layer moduli in FBS layers from data inventory



8 Conclusions

The objective of this research project was to provide guidance to the wider transport industry, based on actual performance data from stabilised pavement solutions in New Zealand, on the characterisation and effective use of stabilised granular materials in transportation projects. The research team assembled into a comprehensive data inventory performance data from New Zealand road pavements that utilised cement and foamed bitumen/cement-stabilising agents, and different material types.

We then used the data in the inventory to investigate the characterisation and performance of pavements incorporating stabilised basecourse materials based on material strength outcomes (eg ITS and UCS), visual inspection and maintenance records, and FWD testing.

The research team came to the following conclusions:

- Near-surface basecourse aggregate materials can be stabilised with cement (incorporating the existing seal layers and/or make-up metal as required) to produce an effective new pavement layer capable of carrying design traffic loads without premature cracking.
- When using cement additive contents of between 1% and 3% by dry mass, the resulting cement-stabilised pavement layer should be modelled as a lightly bound material.
- The design of pavements incorporating lightly bound cement-stabilised basecourse layers should continue to use the guidelines provided in Austroads (2008) and Transit NZ (2007) and the conceptual pavement model developed from this research to optimise design outcomes.
- The conceptual pavement model developed from this research utilises Indirect Tensile or Flexural Beam Testing of samples from the site (collected by milling the in-situ materials with some imported aggregate, as required, to replicate the future stabilisation process) to define material/site-specific modulus, traffic loading and strain constraints that can then be used by the pavement designer to mitigate the risk of premature fatigue in pavements using lightly bound materials.
- When using lightly bound stabilised layers, the maximum tensile stress within the stabilised layer should be <50% of the flexural strength (or less than the tested ITS) of the cemented material, to mitigate the risk of premature fatigue.
- Stabilisation with low cement contents (<1%) will be difficult to achieve in practice. The design of such layers should assume unbound behaviour, unless tested otherwise.
- Stabilisation with higher cement contents (>3% by dry mass) can be expected to deliver a bound material outcome. Design should be in accordance with current Austroads rules, to mitigate the risk of fatigue failure in the new bound layer.
- The design of pavements incorporating FBS basecourse layers should continue to use the guidelines provided in Austroads (2008) and Transit NZ (2007), and include a check that the modular ratio of the FBS layer to the underlying sub-base layer is no more than 5.
- Ongoing refinement of the conceptual pavement model for pavements with cement-stabilised basecourse layers presented herein using data collected from new and existing project sites is recommended.
- Ongoing collection of performance data for pavements incorporating a FBS basecourse layer should be used to develop a conceptual pavement model for pavements incorporating an FBS basecourse.

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Appendix A Laboratory and field test results

INDIRECT TENSILE TEST
TEST REPORT

OPUS

Project:	NZTA Research Project TAR 09/10
Client:	Opus - Napier Office
Contractor:	Not Stated
Sampled By:	Client
Date Sampled:	Not Stated
Sampling Method:	Drilled Core
Date Received:	13/04/11
Sample Condition:	As Received
As sampled description:	Pavement cores

Project Number:	S-24697.IR
Lab Ref No:	SEC11/NA/001
Client Ref No:	006-016/11
Client Job No:	-

Test Results

Lab Ref No	Height (mm)	Diameter (mm)	Mass (g)	Load (P, N)	Load (P, KN)	ITS (Kpa)	Moisture (%)	Bulk Density (t/m3)	Dry Density (t/m3)
006/11	Site 2, Foam Bitumen Stabilised/Cement (2007)								
	130.8	193.1	7597.8	11600	11.600	292.4	1.4	1.983	1.955
007/11	Site 3, Cement Stabilised (2008)								
	121.0	193.0	6722.6	4600.0	4.5	125.4	4.7	1.899	1.814
008/11	Site 3, Cement Stabilised (2008)								
	106.3	193.7	6834.2	3700.0	3.7	114.4	3.7	2.182	2.104
009/11	Site 3, Dacite Cement Stabilised (2008)								
	130.3	191.6	7666.8	10450.0	10.5	254.7	3.1	1.951	1.892
010/11	Site 7, Cement Stabilised (2010)								
	97.0	194.3	5574.4	7500.0	7.5	253.3	1.9	1.938	1.901
011/11	Site 9, CONTROL SITE								
	109.4	133.3	6882.0	16800.0	16.8	506.0	2.7	2.145	2.089
012/11	Site 10, Cement Stabilised (01/02)								
	83.0	194.6	4872.4	11500.0	11.5	453.3	2.1	1.974	1.934
013/11	Site 12, Cement Stabilised (2008)								
	95.0	194.5	6064.8	5300.0	5.3	182.5	1.1	2.149	2.126
014/11	Site 14, Cement Stabilised								
	130.0	140.0	3966.0	6500.0	6.5	227.4	1.2	1.982	1.958
015/11	Site 14, Cement Stabilised								
	69.3	144.2	2395.9	9300.0	9.3	592.5	2.0	2.117	2.076
016/11	Site 15, Cement Stabilised (97/98)								
	113.8	144.2	3518.6	8200.0	8.2	318.1	2.4	1.893	1.849

Test Method: Following that as described in NZS 3112 : Part 2 : 1986, Test 8

Date tested: 13-20/5/11
Date reported: 23/05/11

This report may only be reproduced in full

Approved: Dave Hotham
Designation: Assistant Laboratory Manager
Date: 24/05/11

LAF018 (11/10)

Page 1 of 1

UNCONFINED COMPRESSIVE STRESS WITH YOUNG'S MODULUS

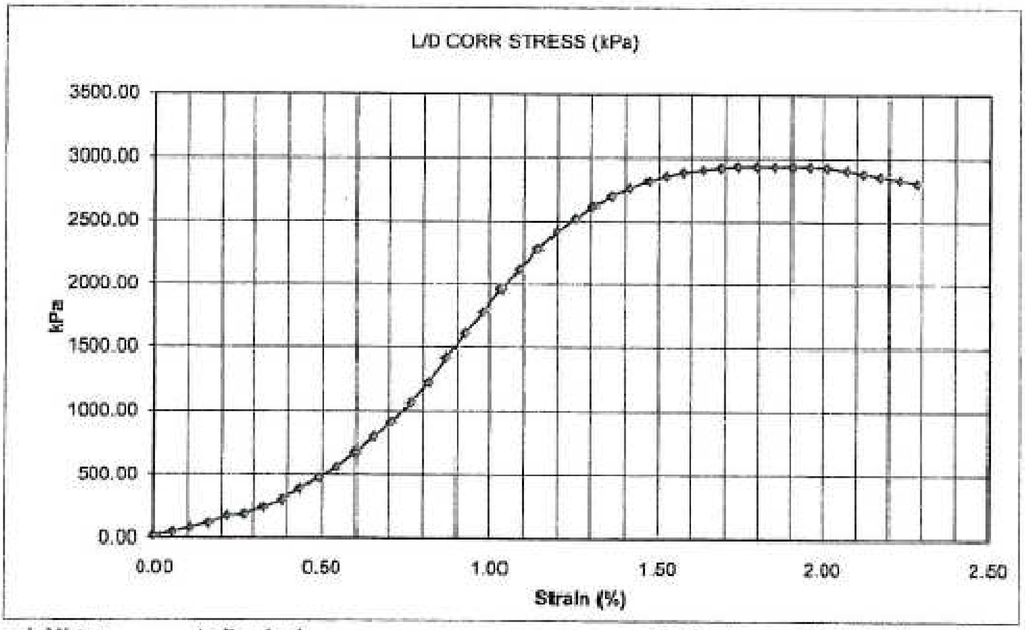
OPUS

Project : NZTA Research Project TAR 09/10

Client : Opus - Napier Office
Client ref : File 2U4697.IR
Sampled by : Client
Date Sampled : Unknown   Date received : 13/04/11
Sampling method : Drilled Core
Sample condition : As received
Sample description Foam Bitumen/Cement Site 1

Project No: 2-S4697.IR
Folder No: SEC11/NA/001
Lab Test No : 001/11

Sample ID	001/11					
Bulk Density t/m³	2.05					
Water Content %	4.6					
Dry Density t/m³	1.96					
Max Stress kPa	2939.32	Young's Modulus		318.3		Mpa
Strain at Failure %	2.0	for Strain		0.71 -1.09		%



Sample History   As Received

Testing is covered by IANZ Accreditation
This report may only be reproduced in full

Test Methods
Unconfined Compression Test NZS 4402;1986 Tests 6.2.1

Tested By: HT & AJ   Date : 13/05/11

19 laboratory

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

IANZ Approved Signatory   Date : 24/05/11
Dave Hotham
Assitant Laboratory Manager

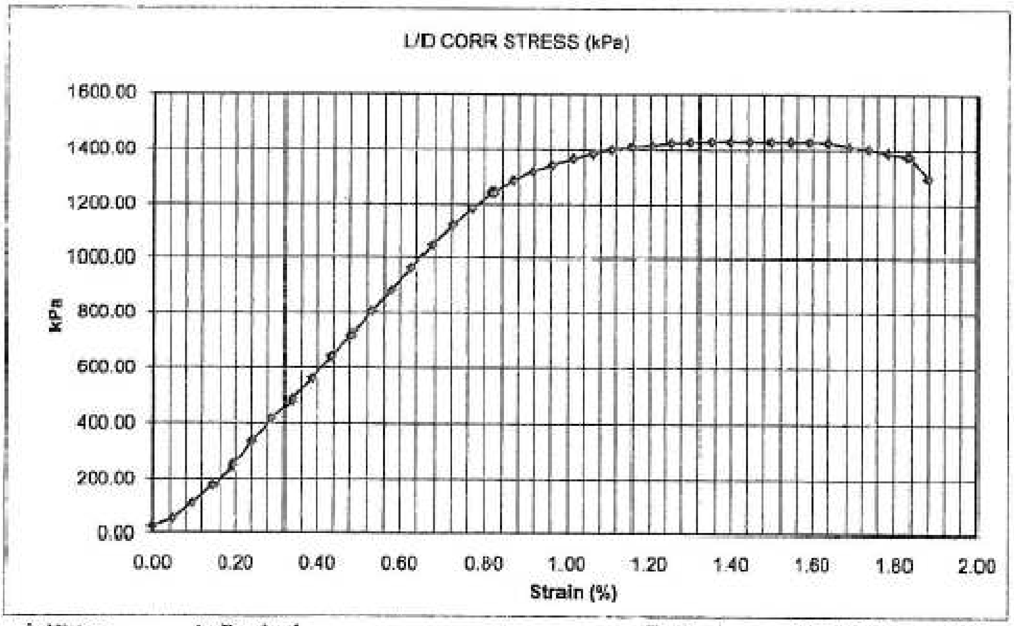
UNCONFINED COMPRESSIVE STRESS WITH YOUNG'S MODULUS

Project : NZTA Research Project
TAR 09/10

Client : Opus - Napier Office
Client ref : File 2U4697.IR
Sampled by : Client
Date Sampled : Unknown Date received : 13/04/11
Sampling method : Drilled Core
Sample condition : As received
Sample description Foam Bitumen/Cement
Site 5

Project No: 2-S4697.IR
Folder No: SEC11/NA/001
Lab Test No : 002/11

Sample ID	002/11					
Bulk Density t/m³	1.93					
Water Content %	5.9					
Dry Density t/m³	1.83					
Max Stress kPa	1432.31	Young's Modulus		163.7		Mpa
Strain at Failure %	1.6	for Strain		0.16 -0.82		%



Sample History As Received

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Test Methods
Unconfined Compression Test NZS 4402;1986 Tests 6.2.1

Tested By: HT & AJ Date : 13/05/11

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

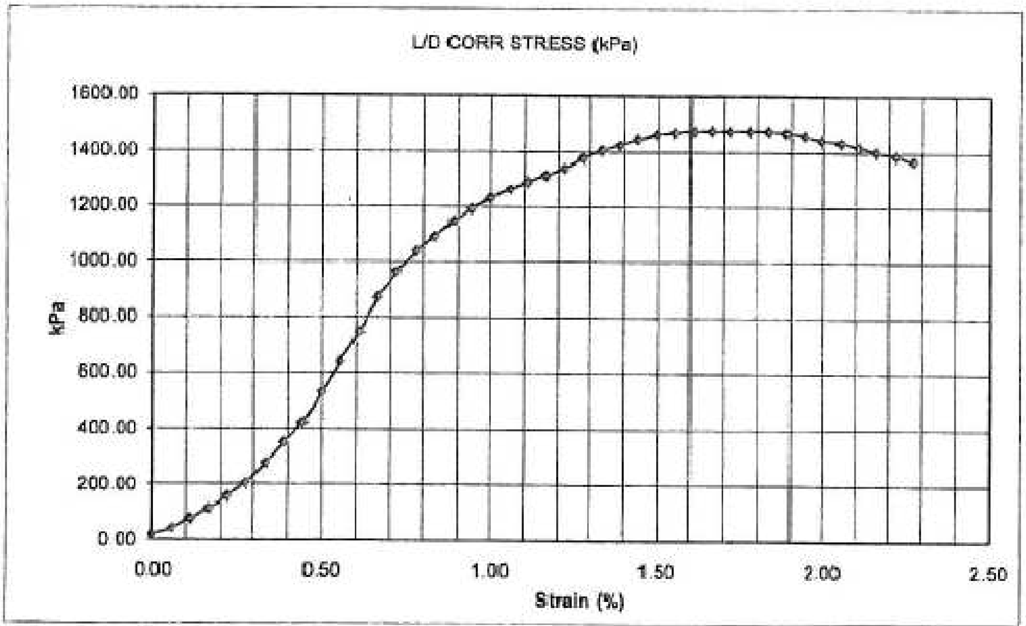
IANZ Approved Signatory Date : 24/05/11
Dave Hotham
Assitant Laboratory Manager

UNCONFINED COMPRESSIVE STRESS WITH YOUNG'S MODULUS

Project :	NZTA Research Project TAR 09/10
Client :	Opus - Napier Office
Client ref :	File 2U4697.IR
Sampled by :	Client
Date Sampled :	Unknown Date received : 13/04/11
Sampling method :	Drilled Core
Sample condition :	As received
Sample description	Cement Stabilized Site 11

Project No:	2-S4697.IR
Folder No:	SEC11/NA/001
Lab Test No :	003/11

Sample ID	003/11						
Bulk Density t/m³	2.17						
Water Content %	2.4						
Dry Density t/m³	2.12						
Max Stress kPa	1476.84	Young's Modulus		203.2		Mpa	
Strain at Failure %	1.8	for Strain		0.44 -0.67		%	



Sample History   As Received

Testing is covered by IANZ Accreditation
This report may only be reproduced in full

Test Methods

Unconfined Compression Test   NZS 4402;1986 Tests 6.2.1

19

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Tested By: HT & AJ   Date : 13/05/11

IANZ Approved Signatory   Date : 24/05/11

Dave Hotham
Assitant Laboratory Manager

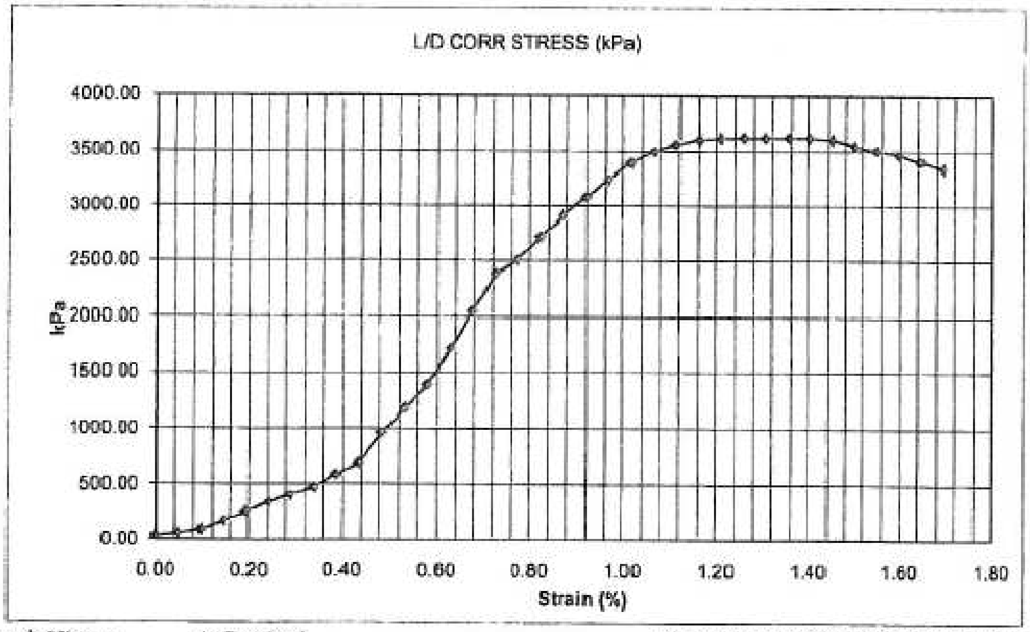
UNCONFINED COMPRESSIVE STRESS WITH YOUNG'S MODULUS

OPUS

Project :	NZTA Research Project TAR 09/10
Client :	Opus - Napier Office
Client ref :	File 2U4697.IR
Sampled by :	Client
Date Sampled :	Unknown Date received : 13/04/11
Sampling method :	Drilled Core
Sample condition :	As received
Sample description	Cement Stabilized Site 12

Project No:	2-S4697.IR
Folder No:	SEC11/NA/001
Lab Test No :	004/11

Sample ID	004/11					
Bulk Density t/m³	2.21					
Water Content %	2.6					
Dry Density t/m³	2.15					
Max Stress kPa	3620.13	Young's Modulus		379.2		Mpa
Strain at Failure %	1.4	for Strain		0.77 - 0.97		%



Sample History   As Received

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This report may only be reproduced in full

Test Methods

Unconfined Compression Test NZS 4402:1986 Tests 6.2.1

Tested By: HT & AJ   Date : 13/05/11

19 laboratory — All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

IANZ Approved Signatory   Date : 24/05/11

Dave Hotham
Assitant Laboratory Manager

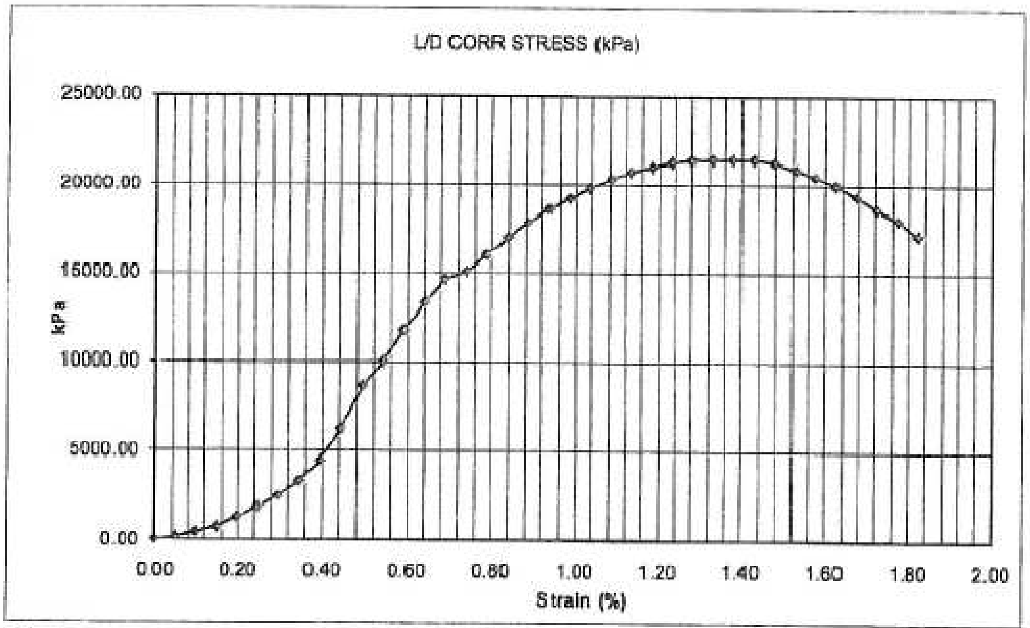
UNCONFINED COMPRESSIVE STRESS WITH YOUNG'S MODULUS

Project : NZTA Research Project TAR 09/10

Client : Opus - Napier Office
Client ref : File 2U4697.IR
Sampled by : Client
Date Sampled : Unknown Date received : 13/04/11
Sampling method : Drilled Core
Sample condition : As received
Sample description Cement Treated Basecourse
Site 13

Project No: 2-S4697.IR
Folder No: SEC11/NA/001
Lab Test No : 005/11

Sample ID	005/11					
Bulk Density t/m³	2.37					
Water Content %	4.2					
Dry Density t/m³	2.27					
Max Stress kPa	21410.98	Young's Modulus		3058.5		Mpa
Strain at Failure %	1.4	for Strain		0.49 -0.69		%



Sample History As Received

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Test Methods
Unconfined Compression Test NZS 4402;1986 Tests 6.2.1

Tested By: HT & AJ Date : 13/05/11

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

IANZ Approved Signatory Date : 24/05/11
Dave Hotham
Assitant Laboratory Manager

Napier Laboratory
90 Prebensen Drive
Private Bag 6019, HB Mail Centre,
Napier 4142, New Zealand

Tel +64 6 833 5590
Fax +64 6 835 3892

TO	William Gray
COPY	
FROM	Jason Crichton
DATE	10 November 2010
FILE	2-S4697.IR / 032 NL
SUBJECT	NZTA Research Project - pavement cores

William,

- The test method for the indirect tensile testing was as directed by you, at 40kN per minute and based upon NZTA draft specification T/16 2010.
- Bulk density was determined by immersion in water test method, NZS 4402 : 1986 test 5.1.4.
- The core samples were dry cored except for site 18 which was water cored.
- Test core diameter and length reported are averaged values
- Each core was given a visual description where possible in accordance with the NZ Geomechanics guidelines.
- The balance of the cores that were untested were returned to you for your reference.

Attached: Indirect tensile report
Core descriptions
CD of core digital images.

Regards

Jason Crichton

INDIRECT TENSILE STRENGTH TEST REPORT

Project :	**Research Project**
Location :	**Various stabilised road locations**
Client :	**NZTA**
Sampled by :	**Downer EDI Works - Auckland Laboratory**
Date sampled :	**October 2010**
Sampling method :	**Diatube core**
Sample description :	**Cored stabilised pavement**

Project No :	**2-S4697.IR / 032NL**
Lab Ref No :	**N10.939**
Client Ref No :	

Test Results

SITE	Bulk density (t/m^3)	Length (mm)	Diameter (mm)	Force (kN)	Indirect Tensile Strength (MPa)
2, core 2	2.33	180.0	190.0	24.1	**0.449**
5, core 2	2.21	207.9	192.5	8.1	**0.129**
10	2.32	124.4	196.1	3.5	**0.091**
9, core 1	2.27	226.1	194.6	18.3	**0.265**
8, core 2	2.34	181.4	194.0	17.3	**0.313**
18	2.43	249.2	194.2	186	**2.447**
7, core 1	2.36	236.2	192.9	10.5	**0.147**
6, core 2	2.29	233.5	192.3	16.8	**0.238**
4, core 2	2.29	239.6	193.3	7.9	**0.109**
3, core 2	2.22	223.0	194.0	7.0	**0.103**
17, core 3	2.29	260.9	194.6	32.2	**0.404**
11, core 1	2.35	175.5	194.3	34.3	**0.640**
12, core 2	2.30	128.0	194.6	7.0	**0.179**
13	2.24	263.7	194.7	30.3	**0.376**
19	2.23	206.2	195.2	13.6	**0.215**
20	2.30	237.8	141.5	39.8	**0.753**
21	2.28	169.5	150.8	13.2	**0.329**

Test Method

NZ geomechanics
Determination of the density of soil, NZS 4402, 1986, test 5.14

Date tested : 27/10/10 to 04/11/10
Date reported : 04/11/10

Approved

J Crichton
Designation : *Assistant Laboratory Manager*
Date : 10/11/10

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Fax +64 6 835 3892

TO	William Gray
COPY	
FROM	Jason Crichton
DATE	10 November 2010
FILE	N10.939
SUBJECT	Research project - Core descriptions

Site two core two

18mm chip seal (grade 4?), dry, tightly packed, max size 25mm sub angular to sub rounded. Split in half at failure

Site 5 core 2

19mm chip seal (small grade 4?), wet, tightly packed, max size 15mm, sub angular
Core was cracked to 150mm from seal surface prior to testing.
Core broke from base at failure.

Site 10

Best remnant of the two taken for testing. The other cores were rotary hoed upon extraction.
25mm chip seal (grade 3/5?), wet, and tightly packed, angular to sub angular, max size 20mm.
Core was wet and disintegrated, crumbled upon testing.

Site 9, core 1

15mm chip seal (grade 3/5?), dry, tightly packed, sub angular to sub rounded, max stone size 25mm, 20mm average, appears well cemented.
Core split in half at failure. Bitumen dispersed through it, very dry, finely graded and cemented to 70mm.

Site 8 core 2

Field note – wet at core interface when cored i.e. zone where core broke off.
15mm chip seal (grade 3/5?) max stone size 20mm, average size 10mm.
Core visually wet but bound at base.
Core split in half at failure - visually wet inside

Site 7 core 1

25mm chip seal (grade 5?), max stone size 30mm, average 20mm.
Core broke from base – wet visually inside.
Core shows original sea (25mm) and AP30 angular base course at base.

Site 6 core 2

15mm chip seal (grade 5?), max stone size 25mm, average 15mm, wet, tightly packed.
Visually – good dispersment of stone and bitumen throughout.
Core broke from base, wet inside.

Site 4 core 2

15mm chip seal (grade 5?), max stone size 20mm, average 10mm.
Core visually wet and had shear cracks prior to testing – possibly from drill action.
Core broke along original cracks and was wet inside.

Site 3 core 2

20mm chip seal (grade 3/5?), seal surface to 60mm – fine (5mm) and visually strongly cemented. Max stone size 20mm to 10mm average. Core split in half upon failure but "crumbly". Bitumen well dispersed through core.

Site 17 core 3

Field note: - core location SH 2/ 678/ 10.665 Outer wheel track, sampled 12/10/10.
14mm seal thickness (grade 5?), max stone size 25mm, average 15mm, sub angular. Sample split in half at failure and was moist inside with some bitumen evenly dispersed through it.

Site 11 core 1

Field note: - Location SH 2 / 592 / 14.230 LHS, outer wheel track (OWT). Near longitudinal and transverse cracked pavement. Core sheared off at wet interface.
Visually appears heavily cemented.
20mm chip seal (grade 3/5?), max stone size 35mm, average 15mm. Core split in half at failure

Site 18

Field note: - Location SH 2 / 678 / 11.735 LHS, outer wheel track.
Sample was water cored due to hardness. Located at extensively longitudinal and transverse cracked zone.
25mm chip seal (grade 3?).
Visually appears to be a dry concrete. There is visually no dispersement of seal through the core. Max stone "greywacke" size 15mm to 10mm average, sub angular to sub rounded.

Opus International Consultants Limited

Base of core shows old seal layer (45 -50mm thickness).

Site 12 core 2

Field note – location SH 2 / 608 / 5690 LHS, OWT. Core sheared off at wet interface
18mm seal thickness (grade 3/5?), max stone size 22mm, average 10mm, sub angular to sub rounded.
Wearing seal surface to 45mm – visually heavily cemented then "crumbly". Split in half at failure, "crumbly"

Site 13

Field note – location SH 2 / 608 / 15825 LHS, OWT.
25mm chip seal (grade 3/5/), max stone ("greywacke") size 15mm, average 10mm, bitumen well dispersed through core. Core split in half at failure

Site 19

25mm chip seal thickness (grade 3?), max stone size 25mm, average 15mm.
25 to 70mm – visually dry and heavily cemented and fine. Sample split in half but "crumbly" at failure

Site 20

Field note – mid block RHS core 1.
22mm chip seal (grade 3?), dry, tightly packed, max stone size 20mm, average 10mm, sub angular to sub rounded. Sample split in half at failure. 25 to 70mm – visually heavily cemented and fine.

Site 21

12mm chip seal (grade 5?). cement modified limestone, max size 20mm, angular.
Split in half upon failure, moist with some bitumen dispersement

INDIRECT TENSILE STRENGTH TEST REPORT

Project :	**Research Project**
Location :	**Port Road - Gisborne**
Client :	**NZTA**
Sampled by :	**Opus Laboratory - Gisborne**
Date sampled :	**03/11/10**
Sampling method :	**Diatube core**
Sample description :	**Cored stabilised pavement**

Project No :	**2-S4697.JR / 032NL**
Lab Ref No :	**N10.1046**
Client Ref No :	

Test Results

SITE	Bulk density (t/m^3)	Length (mm)	Diameter (mm)	Force (kN)	Indirect Tensile Strength (MPa)
Gisborne Port Core 5	2.33	258.0	150.5	36.6	0.600

Test Method

Determination of the density of soil, NZS 4402, 1986, test 5.14

Date tested : 19/11/10
Date reported : 19/11/10

This report may only be reproduced in full

Approved

J Crichton

Designation : *Assistant Laboratory Manager*
Date : 19/11/10

Appendix B Conceptual model for cement-stabilised basecourse

B1 Interim mathematical formulation for cement-stabilised basecourse

The following process has been followed for the interim model:

- Deduct the ESA recorded in the first year (N_c, during curing) from inventory data, then refer to the post-curing fatigue life, N_f, ie $N = N_c + N_f$.
- $N_c = N_f / k$ (to be refined sometime, but adopting k=25), which implies that if the pavement is not stressed to a high level, then traffic during the curing period contributes much less to fatigue, as it can heal.
- From inventory data, infer that the rate of decrease of E/E_c at any point where the tensile strain under a 1ESA load is relatively invariant, is initially slow during the end of curing (from 1–2 years) and then increases; ie explore a power law for modulus decay of the form $E / E_c = 1 - (a * N_e)$ ^ b, where N_e is the number of ESA from the end of the curing period to the time of FWD testing. Note: this would also encompass a linear model if b=1.
- In practice, as E decreases there will be lesser load spread and therefore the tensile strain will increase further, because of lesser load spread as well as modulus reduction. This will need to be investigated further once more data becomes available to allow refinements.
- Carry out in-situ determination of moduli and layer 1 strain, both in the wheelpath and also just outside the wheelpath.
- Subsection pavement into uniform treatment lengths.
- Define $E_{(t,N,\ \varepsilon_t)}$ to be the modulus at year t after N ESA and ε_t is the tensile strain at the bottom of the stabilised layer.
- Assume that because a uniform treatment length is identified, the range of ε_t is relatively small.
- Determine the average characteristic (log basis) for $E(t_m,N_m,\varepsilon_m)$ and $E(t_m,1,\varepsilon_m)$ and corresponding average tensile strains (t_m is time in years at measurement, N_m is the ESA repetitions reached at time of measurement). Calculate the ratio of these two moduli.
- From figure 5.12, read off for time t, the value of $E(t_m,1,\varepsilon_m) / E(t_1,1,\varepsilon_m)$.
- Solve the above, for $E(t_m,N_m,\varepsilon_m) / E(t_1,1,\varepsilon_m)$.
- Determine the reference modulus for this test point as $E_c = E(t_m,N_m,\varepsilon_m) / [\ E(t_m,N_m,\varepsilon_m) / E(t_1,1,\varepsilon_m)\ ]$.
- Using the reference modulus, recalculate the reference tensile strains (εtc) at the base of the layer under a 1ESA load, and plot these on the E_c vs ε_{tc} graph.
- From the point with E_t, the above coordinates project forward along the decay line until the fully cracked E_b modulus is reached. That gives the end of life from fatigue. Additional life as unbound could then be considered, if appropriate.
- Iterate with progressive trials for the variables (coefficients of the decay power law and fatigue relationship) until the model fits the observed data.

B2 Interim mathematical formulation for FBS material

This model is to be structured once sufficient observed performance data is available – at least two or three sites will need to be approaching a terminal distress condition.